Petroleum Directorate
Government of Newfoundland and Labrador

OFFSHORE INSTALLATIONS:

GUIDANCE ON DESIGN

AND CONSTRUCTION

Final Draft, August 1982

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FORWARD

This document has been prepared to explain the procedure whereby fixed and mobile offshore installations are certified as being fit for their purpose in accordance with the Offshore Installations (Design, Construction and Survey) Regulations, 1982.

The document has been sub-divided into four principal parts.

Part I deals with certification procedure, the responsibilities of owners and certifying authorities and the outline and purpose of major, annual and additional surveys.

Part II is divided into eight sections presented in the sequence followed by the technical requirements embodied in the Regulations. Sections 2 to 7 are concerned primarily with civil and structural engineering and associated specialist technologies, linked by Section 8 to Parts III and IV, which deal respectively with electrical and mechanical engineering.

Technical developments are still taking place, lessons are being learned in the light of experience and research projects set up several years ago are coming to fruition. New or amended pages will be issued as and when it becomes possible.

The Petroleum Directorate acknowledges the generous help received from the oil companies and the offshore industry, from the certifying authorities, from professional and scientific bodies and from numerous other organizations and individuals. In these guidance notes Minister refers to the Minister in charge of the Petroleum Directorate.

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PART I CERTIFICATION Part I Section 1

SECTION 1 PRINCIPLES

SUMMARY

Section 1 contains general information on the certification scheme set out in the Offshore Installation (Design, Construction and Survey) Regulations, 1982, which for brevity will be referred to as the Regulations, and deals in greater detail with the independent design assessment required under Section 5(2) of the Regulations. Section 2 deals with subsequent certification actions, namely, the surveys and inspections to be undertaken during construction and in service under ~~Regulations~~ sections 5 (2) (b), 6, 7 and 8 of the Regulations.

Matters already covered adequately in the Regulations and which do not require interpretation or further amplification are not repeated in these notes.

1.1 HISTORY

In accordance with the terms of the Petroleum and Natural Gas Act, Chapter 294 of the Revised Statutes of Newfoundland, 1970 the Legislature of the Province of Newfoundland and Labrador has enacted the Offshore Installation (Design, Construction and Survey) Regulations 1982 to provide the procedure for certification of offshore installations in Newfoundland and Labrador waters.

These regulations cover any installation, whether floating or not, which is used in connection with the production of oil or natural gas or associated with the conveyance by means of a pipe constructed in or under the sea.

Any registered vessel amongst the units covered by the above definition will also remain subject to appropriate national flag merchant shipping rules.

1.2 CERTIFICATION SCHEME

The Offshore Installations (Design, Construction and Survey) Regulations 1982 require all offshore installations established or maintained in Newfoundland and Labrador waters to be certified as fit for the purposes specified. These regulations gave statutory force to the practice long followed by responsible engineers in this Province, namely, ensuring that all aspects of the design and construction process are

subjected to critical scrutiny by an independent professional eye.

The following organizations have been authorized to issue Certificates of Fitness after satisfying themselves that it is proper to do so.

American Bureau of Shipping
Bureau Veritas
Det Norske Veritas
Lloyd's Register of Shipping

In addition the Minister may himself issue Certificates of Fitness if he should see fit to do so.

1.3 INSTALLATIONS COVERED BY THE CERTIFICATION SCHEME

The Offshore Installations (Design, Construction and Survey) Regulations 1982, cover all installations including mobile and fixed drilling and production installations, accommodation units, oil storage units, loading units, flare stacks, installations supporting pipeline manifold or booster facilities, pipelines and subsea systems and any other type of installation which is concerned with exploration for, exploitation of, or associated with the conveyance by pipeline of petroleum products. An installation manned only on occasion will be required to comply with those regulations which are appropriate.

Where a number of units form a functional group, say, connected by permanent bridges, they may either be considered as a single installation or as separate installations as owner and certification authority may jointly decide.

A structure, and its associated equipment, established below water and accommodating equipment remotely controlled from an offshore installation should normally be considered to be a part of that installation and will be within the scope of all relevant clauses in this publication. Offshore loading units which are normally unmanned could be considered as a separate installation.

1.3.1 INSTALLATION/EQUIPMENT

Equipment deemed to form part of an installation:

(a) Production Installations. Any part of the equipment from the subsea safety valve upward should be treated as part of the installation. Where principal well control equipment is situated on the seabed the Christmas tree, its control system and actuators, subsea manifolds, any devices to protect against physical damage and any other equipment will form part of the installation.

The seabed connection for a disconnectable riser, eg. for a floating production installation or any similarly placed manifold is part of the installation.

(b) Drilling Installations. Any part of the equipment from the casing head, including the marine riser and blow-out preventor, (BOP) should be treated as part of the installation.

(c) Pipelines could be considered as a separate installation.

(For the purposes of this section a Christmas tree should be taken as being an assembly of valves and fittings used for production control and includes all equipment down to the tubing head top flange. A blow-out preventer should be taken as a unit attached to the casing or to a unit of wellhead equipment installed on the casing for the purpose of controlling pressure in the annular space between the casing and an inner string of pipe during drilling, completion or workover operations).

1.3.2 INSTALLATION/EQUIPMENT NOT COVERED BY THE CERTIFICATION SCHEME

Dredging installations registered as vessels are not covered by the Design, Construction and Survey Regulations. Further, the Certification Scheme does not cover crane barges, pipe-laying craft and standby vessels.

1.3.3 INTERFACE BETWEEN A PIPELINE AND ANOTHER INSTALLATION

1.3.3.1 Introduction

In order to avoid unnecessary duplication of assessment, the following paragraphs explain the general principles for defining the interface between pipeline and another installation, and the manner of obtaining agreement to the proposed interface in a particular case.

The interface should be on a "case-by-case" basis. However, the following paragraphs outline the general principles which should be adopted to define the interface.

The objectives for the pipeline instrumentation control and ESD equipment and the assurance that these objectives are being achieved would be made available during the certification of the pipeline.

1.3.3.2 Definition of Pipeline

The pipeline will extend 50m from the start of the seabed section of the pipeline and all pipe work risers and valves and fittings up to the 50m mark would form part of the associated installation.

At subsea wells, the pipeline will normally terminate 50m from the well Christmas tree pull-in connection or wing valve.

At an underwater installation where none of the above definitions is applicable, the pipeline will terminate 50m from the pull-in connection.

1.4 DATE OF CERTIFICATION

The regulations stipulate that:

(a) no fixed installation may be established in the relevant waters, as defined in the Petroleum and Natural Gas Act 1970;
(b) no mobile installation may be brought into those waters; and
(c) no fixed or mobile installation may be maintained in those waters;

unless a valid Certificate of Fitness is in force in respect of that installation. But a mobile installation may enter the relevant waters, however, if bound for a port or roadstead for repair, alteration, or to enable certification to take place.

Fixed drilling and production installations pose problems because they arrive at their final location in a variety of

stages of completion; from the bare skeletal framework of the steel jacket to an almost complete installation of the concrete gravity type. The point in time at which a fixed installation requires a Certificate of Fitness is petroleum operation related. Exploration and production installations require a Certificate of Fitness at the commencement of drilling operations, defined by the Petroleum Directorate as that moment at which a well control mechanism is established.

Manifold platforms, booster platforms, storage units and similar types of installation require a Certificate of Fitness before hydrocarbon flow begins.

Accommodation units for personnel associated with the above operations must also comply with the Regulations and require a Certificate of Fitness before they are commissioned for use.

In accordance with Section 9 (5) of the Regulations, a Certificate of Fitness shall be valid for such period as the certifying authority may specify, not exceeding five years from the date of issue.

1.5 OWNER'S RESPONSIBILITY

Responsibility for applying for a Certificate of Fitness for an offshore installation that is to be established or maintained in the relevant waters and for ensuring the continued validity of that certificate, rests with the owner of the installation. Application should be made in the form specified in the Design, Construction and Survey Regulations and may be addressed to any one of the organizations listed in 1.2 above, at the owner's discretion.

The owner is responsible for providing, or causing to be provided, all technical information necessary to enable the certifying authority to make a comprehensive and independent assessment of the design; for providing continuous access and all facilities required by the certifying authority during construction; and for providing access and any specialist assistance required, and full access to logs and records, during subsequent surveys.

The owner has a statutory responsibility for the sound design, proper construction and effective maintenance of his installation, including adequate supervision and inspection necessary to achieve these ends, but a valid Certificate of Fitness is evidence that an independent and responsible organization believes the owner to have honoured his statutory obligations.

Experience in the certification process in other areas shows it is to the advantage of all concerned to appoint the certifying authority as early as possible in the design stage and prior to commencement of construction. Sound procedures established during the early stages make for smooth running during subsequent overlapping stages of design and construction.

1.6 RESPONSIBILITY OF THE CERTIFYING AUTHORITY

The primary responsibility of the certifying authority is to be satisfied that the measures taken by the owner to comply with his statutory obligations are adequate and effective and, when so satisfied, to issue a Certificate of Fitness.

1.6.1 CONFIDENTIALITY

During the course of its work, the certifying authority will obtain access to valuable information about, for example, design, methods of construction and operation and the results of research. It shall ensure that the applicant's rights to such information are preserved and that the information is not disclosed except to the Minister, the Petroleum Directorate or any person duly authorized to act on behalf of the Minister.

1.6.2 CONSULTATION WITH THE MINISTER

In the event of a difference arising on the application of the regulations which cannot be resolved between the owner and certifying authority, the certifying authority should, at the formal request of the owner, refer the matter to the Minister with an agreed precis of the points of difference.

1.6.3 VESSELS REGISTERED IN CANADA

In the case of an installation which is registered in Canada as a vessel, arrangements should be made with the Canadian Coast Guard for interchange of appropriate documents in order to secure compliance with the requirements of merchant shipping law.

1.6.4 INDEPENDENT ASSESSMENT OF DESIGN

1.6.4.1 Objective

The primary purpose is to ensure that all planning criteria and all vital elements of design are brought under independent

critical scrutiny, to eliminate errors arising from incorrect or unjustified assumptions associated with loading, stress analysis, material properties, foundations and structural behaviour; mathematical errors contained in the calculations; and incorrect deductions from the results. Every assessment or other check should be independent in character, separate from the original detailed calculations (but not necessarily using a different method of calculation) and carried out by persons other than those involved in making the original calculations.

The matters to be reviewed are futher outlined below, set out under the headings used in Schedule II of the Regulations.

1.6.4.2 Environmental considerations

The certifying authority should satisfy itself that all environmental reports have been prepared by competent persons and comply with the minimum requirements as specified in the guidance notes and appendices.

For mobile units, it should be ensured that the environmental conditions postulated by the applicant take into account all the relevant requirements for certification. After the check of design environmental loads and predictions of installation behaviour there should be identified, for endorsement on the Certificate of Fitness, the appropriate limitations as to wind, temperature, wave and current conditions, sea ice and icebergs, water depth, and for bottom-supported installations, type of sea-bed. (See also Part II Section 4.2.2.1.) The terms of these limitations should be agreed with the applicant.

For fixed units and for mobile units intended to be stationed for extended periods of time, it should be ensured by scrutiny of the relevant reports that the environmental conditions used in the design are those to be expected at the intended site, and that they take into account all the relevant requirements for certification. An appropriate check should be made as to the effect of sea-bed conditions.

Appropriate Government establishments may be consulted as necessary to establish the validity of the assumptions, methods of extrapolation and prediction, calculations and conclusions as to wind, temperature, waves, currents, sea ice and icebergs contained in environmental reports submitted by the applicant.

1.6.4.3 Foundations

The assessment of foundations should review the vertical and

horizontal loads and their distribution to be transmitted to the sea-bed under the most severe conditions applicable. In the case of a fixed unit the adequacy of the site investigation should be checked and its interpretation should be reviewed. It should be ensured that for all installations that are or may be bottom-supported there are adequate protective measures against scour. (See also 1.7 and Part II Section 4.2.2.1.)

1.6.4.4 Primary structure

The whole of the design process should be subjected to close scrutiny and check, which should include verifying that:

(i) the design criteria, including those of the environment, are clearly stated, rational and sufficiently comprehensive for the intended use and location, or locations, of the unit;
(ii) the assumptions made at all stages of the design process are clearly stated and soundly based;
(iii) possible modes of major failure have been identified;
(iv) design decisions based on model tests have taken full account of all factors governing their application to full-scale units;
(v) calculations are clearly stated, sound and in accordance with good practices;
(vi) materials are adequately specified and suitable in all respects;
(vii) structural steel connections are designed in accordance with good practice, e.g. minimization of stress concentrations and compatibility with efficient welding assembly and inspection techniques; and
(viii) the design evaluation shows that the structure satisfies the initial design criteria in all respects.

A sufficient number of independent strength and stability calculations should be carried out to establish that the calculations contain no mathematical, procedural or other errors. Computer input data should be thoroughly checked and the programmes used well proven. The certifying authority should satisfy itself that all programmes used are suitable for their purposes.

To the maximum extent practicable, new and existing units should be treated alike. Where, however, an existing unit is being certified, the history of the conditions experienced and the behaviour of the unit under those conditions should be taken into account along with the record of damages and repairs. Subject to satisfaction as to all items (i) to (viii) above, there may be submitted for part of the design evaluation as appropriate a comparison of the performance in use against the design assumptions and criteria.

1.6.4.5 Secondary structure and fittings

It should be checked that each part of the structure is in accordance with the relevant standards and complies with requirements in these guidance notes.

1.6.4.6 Materials

It should be verified:

(i) that materials that are to be or have been incorporated in the structure have been properly selected;

(ii) that adequate supervision, inspection and tests will be or have been undertaken to ensure that the materials supplied or manufactured are in accordance with the specification; and

(iii) that the persons or organizations responsible for quality control within the manufacturing unit are reputable, competent and experienced in their duties.

Where the installation is being certified at the time of building, the quality controls devised by the applicant, his contractors and suppliers should satisfy requirements approved by the certifying authority. It should be checked that, prior to incorporation of materials in the structure, adequate precautions are planned, and taken, to limit corrosion or other deterioration, and tnat all necessary information on properties of materials is made known to those concerned with fabrication so that the correct procedures may be adopted.

Where the installation is being certified after completion, a scrutiny should be made of the relevant records of quality control, corrosion, etc, precautions and fabrication procedures used; if records are deficient or any aspect is in doubt it should be further investigated, eg by tests.

1.6.4.7 Construction

Supervision during construction is covered in Section 2 of Part I.

1.6.4.8 Equipment

It should be checked that there are included those items which are required to be incorporated, that all necessary safety features are incorporated and that all equipment is in accordance with these guidance notes or as specified in Part I Section 1.6.6.3 and correctly assembled.

1.6.4.9 Operations Manual - assessment

A check should be made that all relevant information and advice (specified under each section of the Guidance Notes) is included in the Manual and is consistent with the design and construction of the unit. (See Section 1.7 below.)

1.6.5 RENEWAL OF CERTIFICATE OF FITNESS

Paragraphs 1.6.4.1 to 1.6.4.9, above, apply primarily to initial certification. For recertification the check of design and construction need not be repeated but the records of initial certification and of checks in connection with subsequent changes should be reviewed and any aspect needing further investigation at that time (eg fatigue life prediction, weaknesses revealed during service) should be given due attention to ensure that design and related construction continue to satisfy the requirements.

1.6.6 DISCRETION ALLOWED TO CERTIFYING AUTHORITIES

1.6.6.1 Equivalents

The certifying authority has discretion to accept methods and techniques equivalent to those outlined in this publication subject to being satisfied that no diminution of safety will be involved but the Minister should be informed if the proposal is of unusual or controversial character.

1.6.6.2 Specialist Reports

When called upon to deal with highly specialized matters, such as certain items of drilling or production equipment, or when an authoritative report has already been made on some matter covered by the certification scheme, the certifying authority may accept a report prepared outside its own organization but in each instance the authority must be satisfied that the person or organization concerned has due competence in the particular field, and that that check is truly independent; and must itself review the report (but not carry out a second check) to ensure that the assumptions, data and results are both comprehensive and otherwise acceptable.

1.6.6.3 Recognized Standards

Where matters of detail are not specifically covered by this

document the certifying authority may recognize any generally accepted standards and practices.

1.7 OPERATIONS MANUAL

1.7.1 PURPOSE

The prime purpose of the operations manual is to draw to the attention of the installation manager (person in charge), and other persons, that essential information which is required to enable him to discharge his responsibility to ensure that operations and maintenance are carried out in such a way as to ensure as far as possible, the safety of persons on the installation and the safety and integrity of the installation.

The operations manual for an offshore installation, as required by the Offshore Installations (Design, Construction and Survey) Regulations 1982, should contain written particulars of the installation for the information, guidance and instruction of the installation manager to secure, in the case of a fixed installation, the safety of the installation when established at a station and, in the case of a mobile installation, the safety, seaworthiness and stability of the installation when moving to or from, being located at, removed from, or maintained at, a station. It should also contain particulars of any observations that should be made and the circumstances in which reports should be made to the owner and certifying authority. All information should be provided in a manner which is convenient for use and unambiguous in meaning.

Operations and procedures covered by other regulations should be identified but need not be included in the operations manual but the operations manual may, should the owner so desire, form a separate self-contained section in a more comprehensive manual dealing with other duties and matters under the control of the installation manager. However, all efforts should be made to make the operations manual as concise as is reasonably practicable, and it should be suitably indexed to ensure quick retrieval of the information that it contains. It should be submitted to the certifying authority for their consideration and approval prior to the issue of the first certificate of fitness and for review by the certifying authority prior the recertification. It should be amended as necessary to keep it up to date. All amendments should be approved by the certifying authority, and should be recorded in the operations manual.

1.7.2 CONTENTS: GENERAL

The operations manual should contain at least the following information (as applicable to that particular installation) and additional relevant information considered necessary by the owner:

(a) Any exemption together with its relevant conditions granted by the Minister.

(b) Any limitations on the operation of the installation, its equipment, eg cranes, or matters concerning the current Certificate of Fitness.

(c) Relevant information as to the environmental considerations and their effect on the installation which may be of use to the installation manager. For fixed installations this should include any environmental conditions which may require evacuation at specified sea ice, iceberg or meteorological forecast, the amount of snow and ice which may be allowed to accumulate on deck, and the marine growth design limitations. For mobile installations additional requirements include the environmental operating limitations, the effect of wind, sea, snow, ice, sea ice, icebergs and marine growth on the strength, stability and seaworthiness for the various modes (ie on location and in transit).

(d) The characteristics of the installation foundation, bottom penetration and limiting values for scour and other sea-bed conditions should be given for fixed installations. Mobile installations which are supported by the sea-bed should have information concerning the different sea-bed conditions acceptable for that installation including the varying capacity of the sea-bed, limiting values of sea-bed slope and maximum penetrations of footings.

(e) Information concerning stability aspects of mobile installations including all data and instruction necessary to enable the installation manager to determine whether any intended configuration of, or change to, the loading or ballasting will satisfy the stability requirements which may apply to the installation.

(f) Relevant information concerning permissible deck loads, variable load limits and preloadings.

(g) Details of any colour coding system.

(h) Information on corrosion protection systems used and any specific guidance on safety and maintenance of the system.

(i) Details of openings and means of closure in watertight compartments called for by the Petroleum Directorate or certifying authority.

1.7.3 CONTENTS: DRAWINGS

The following drawings should be included in the operations manual:

(a) For all installations:
general arrangements of the deck structure, accommdation, helideck, and equipment contained on the topside facilities.

(b) Fixed steel platforms:
general arrangement drawing showing the jacket, piling, risers and conductors.

(c) Gravity structures:
general arrangement drawings showing the lower concrete or steel structure including any skirt arrangements or piling, the deck structure connection to the lower structure, the risers and conductors.

(d) Jack-ups:
general arrangement drawings showing the main and supporting structure, the equipment of elevating and lowering of the deck structure and any arrangements for towing.

(e) Semi-submersibles:
general arrangement drawings showing the main and support structure, the method of maintaining station, ie anchoring system or DP system, and arrangements for towing where fitted.

(f) Drill ship and similar shaped vessels such as tanker oil storage vessels and offshore loading units:
the hull structure, the positioning equipment, ie anchoring system or DP system or others as applicable.

(g) Reasonably detailed drawings should be provided for all or any of the above types of installations to describe any special type of joint or mechanism incorporated in the structure which may determine the integrity of the overall structure.

(h) Detailed drawings of all openings and means of closure which could effect the stability of the installation and drawings of ballast and bilge systems should be included for all cases of installations required to meet stability requirements.

(i) Drawings showing the various hazardous areas on the installation.

(j) Drawings showing accurately the area where any dangerous substances mentioned in the Newfoundland and Labrador Petroleum Drilling Regulations, 1982 may be stored.

(k) Drawings of safety equipment layout showing escape routes, fixed fire extinguishing systems and locations of life-saving appliances.

(l) Details of corrosion protection system.

1.7.4 OPERATIONAL CRITERIA

(a) Where under design considerations advantage has been taken of the capability to readjust loads on the installation, by suspending operations or by other measures within the control of the installation manager, and for floating units the capability of altering the loading, trim, draught, orientation or mooring systems, clear instructions should be provided in the operations manual, including limitations on ballast operations pertinent to acceptable loading conditions for the installation, eg adjustment of drawdown in gravity structures.

(b) The maximum helicopter weight and wheel centres for which the platform has been designed and the maximum size and weight of the helicopter for which the deck is certified should be given. The extent of the obstacle free area for the helicopter requirements should be given.

(c) Special arrangements or facilities for the inspection and maintenance of the installation's primary and secondary structures, any equipment or plant, and any crude oil storage facilities on or in the installation should be clearly described.

(d) Reference should be made to special precautions or considerations to be employed when repairs or alterations to the structure are to be undertaken.

(e) Any special operational requirements or emergency requirements covering essential features, eg details of the procedure on loss of pressurisation to spaces maintained at an over pressure to prevent ingress of explosive gas/air mixtures should be clearly described.

(f) Relevant information concerning permissible deck loads, variable deck load limits and preloading should be clearly stated.

1.7.5 OTHER CONSIDERATIONS

The operations manual should, with regard to possible emergencies and action to be taken during emergency operations and return to normal operations, make reference to the emergency procedures manual.

Suitable entries should be made regarding the type and extent of any material used in the construction of the installation which may be hazardous to the health of persons required to carry out any repairs on such materials. Suitable precautions to be complied with when working with these materials should also be given.

A description of the equipment for platform elevation and lowering and special types of joints should be given and details of their purpose, proper operation and maintenance

should be included in the interest of the safety of the installation.

Brief particulars of the major systems on the installation including simplified flow sheets should be included. For the operation and maintenance of equipment on the installation, there should be separate and comprehensive manuals available to the installation manager. These may be scrutinized by the certifying authority during surveys of the platform. Supporting design philosophy and data should be available and readily retrievable at the operator's onshore base.

For ballasting, bilge pumping, dynamic positioning and mooring equipment there should be included additionally, with respect to the range of operating conditions expected, guidance when action may be necessary and instructions on the action to be taken in the installation's various modes of use.

1.7.6 REFERENCES

Reference should also be made to the following:

Part	Section		Page
I	1.6.4.9	Operations Manual - assessment	10
II	2.3	Operations Manual: environmental criteria	40
II	3.1.9	Operations Manual - foundation characteristics	49
II	4.1.6	Loads in combination - extreme conditions	64
II	4.1.7	Loads in combination - operating conditions	65
II	4.2.2.1	Bottom supported units	79
II	4.3.1	Floating intact stability	85
II	5.4	Watertight compartments	105
II	5.8.2	Structural fire protection	111
II	5.10	Escape routes	117
II	5.11	Hazardous areas	118
II	5.12	Facilities for inspection and maintenance	118
II	6.1.2.4	Operations Manual - maintenance	124
II	7.2.4	Operations Manual - precautions	134
II	8.10	Operations Manual - equipment	146
IV	2.1	Piping systems	152

SECTION 2 SURVEYS AND INSPECTIONS Part I Section 2

SUMMARY

This section outlines the nature of the surveys and inspections that should be made during and after construction.

2.1 OBJECTIVES

The surveys described in this section are to ensure that the intentions of the designers are completely and correctly reflected in the installation as constructed and that the installation is thereafter maintained to predetermined standards of strength, stability and, where appropriate, seaworthiness. All surveys are to be undertaken by competent and independent persons.

2.2 GENERAL PROCEDURE

The owner is responsible for arranging for surveys as and when they become due.

In view of the wide variety of installations and the problems of access to critical areas, particularly below water, an agreed schedule of inspections and tests should be drawn up in each case; this schedule should be arranged by owners and certifying authority in consultation and, where appropriate, with the organization responsible for the underwater inspection and testing and any other specialist body whose services will be required. This schedule should be sufficiently flexible to permit adjustments to be made in the light of findings as the survey proceeds.

The certifying authority surveyor should agree particulars of destructive and non-destructive tests to be applied to materials and to workmanship and the number, frequency or circumstances in which tests should be made; he should also satisfy himself as to the competence of the personnel and organizations concerned. He should monitor all tests and request spot checks and confirmatory tests to be made as he judges necessary. Copies of all test reports are to be retained.

Surveys are to include, where appropriate, the scrutiny of all records of service history including logs, reports, test certificates, damage and repairs, reports of inspections and tests carried out by the owner's representatives, the use and interpretation of any instruments installed to monitor

structural behaviour, watertight integrity, foundation conditions and any other factors relevant to the continued safety of the installation, and any other data relative to the condition of the installation and its behaviour in service. Where other current certificates exist, or will exist, surveys for such certificates are, as far as possible, to be combined with those for a Certificate of Fitness.

Following any survey the owner and certifying authority should consult on any repairs or other actions that may be required.

2.3 SPECIFIC SURVEYS

2.3.1 MAJOR SURVEY-NEW CONSTRUCTION: ALL TYPES OF INSTALLATION

The purpose of this survey is to ensure that all work is carried out strictly in accordance with the intentions of the designer as set out in the drawings, specifications and other documents approved by the certifying authority. The initial major survey should be a continuous activity covering the whole of the construction period and the installation and testing of equipment: in the case of fixed structures it will be the last opportunity to inspect and test under good conditions those elements that will be permanently submerged.

Procedures to be followed should be agreed between owner and certifying authority at an early stage, including procedure and timetable for approving drawings and the organization for controlling the quality of materials and workmanship. There should be systems for anticipating, where possible, problems and differences and for resolving them with minimum delay.

It is the duty of the certifying authority to ensure that construction work and the installation of equipment is continuously supervised at all times by competent and independent persons both at the main construction site, or sites, and at the places of manufacture of materials, assemblies and sub-assemblies.

2.3.2 MAJOR SURVEYS-INITIAL SURVEYS OF EXISTING INSTALLATIONS

These surveys, which will have been preceded by a design assessment, will only apply to mobile installations that have not hitherto been certified. They should follow the general lines set out in 2.3.3 below.

2.3.3 MAJOR SURVEYS: RE-CERTIFICATION

The object of this survey is to ensure that any deterioration is within acceptable limits and that the installation continues to comply with all relevant requirements of the Offshore Installations (Design, Construction and Survey) Regulations.

In-service inspections of fixed installations should be planned by an experienced engineer who has examined the design characteristics, the records of severe environmental and other loads to which the structure may have been exposed and any available records of structural behaviour such as settlement, differential settlement, tilt, distortion or abnormal response. The initial inspection schedule should take account of the nature of the deterioration to which steel and concrete structures are liable in a marine environment and of the regions in which defects are most prone to occur (eg sudden changes in section, discontinuities, etc.) and of members or regions known to have been, or likely to have been highly stressed or subjected to severe fatigue loading. Special attention should be given to areas of suspected damage or deterioration and to areas repaired following earlier surveys. Only after the nature and probable positions of defects have been established should the inspection schedules be prepared.

Fixed structures in deep water present special problems in that examination of certain areas may present serious or unacceptable hazards. Subject to the need to ensure the continued safety of the installation the inspection schedule should require deep diving only where no other satisfactory means exist of carrying out essential work. Consideration should be given to using manned or remotely controlled submersibles.

In a jacket-type structure the outer, more accessible, members may reasonably be taken to be representative of the internal members unless the original design or service history suggest otherwise.

The lower elements of a gravity-type or a piled-type structure are likely to suffer serious damage if partly deprived of support by erosion, or by excessive or uneven settlement, and many units of this type should incorporate instruments to record the state of the foundations.

Service history and/or the design assessment will suggest areas requiring special attention and, if no other defects are detected, other deep-water areas need receive only spot checks.

There are grounds for believing that corrosion of concrete reinforcement is less likely in permanently submerged areas

than in the splash zone, quality and other circumstances remaining unaltered. If concrete in the splash zone is found to be in good condition only a limited number of spot checks need be made at lower levels, except at sudden changes of section and other areas of high stress concentration.

Account may be taken of data recorded by instruments installed to monitor structural and foundation behaviour.

All installed equipment should be examined to ensure that it complies with the requirements of Part II, Section 8, and Parts III and IV of this document. Servicing and maintenance records should be examined.

Mobile installations should be moved to a suitable location for a major survey, preferably to a dry dock or alongside in a shipyard, otherwise to sheltered waters. If possible and feasible, surveys may be conducted in-situ.

2.3.4 ANNUAL SURVEYS

The purpose of the annual survey is to ensure that any deterioration of the structure is within acceptable limits; that secondary structures and fittings concerned with the safety of the installation and the safety of personnel are in sound condition; and that mechanical and electrical equipment and installations are being maintained to satisfactory standards.

The second annual survey after each major survey should include a general inspection of major parts of the installation below the splash zone to determine whether any change has occurred in the condition of the installation since the main survey was carried out. The survey should include an inspection of the sea floor scour corrosion protection system, and a thickness determination of the primary structure. The survey should verify that no structural changes have been made since the previous survey.

Annual surveys should include a close visual inspection down to and including the splash zone, or to the water line at maximum freeboard as may be appropriate, to detect obvious damage and indicate areas likely to warrant further investigation. In the light of this inspection, previous service history, etc., the surveyor may then request that further inspections and/or tests be made, above or below water. The surveyor should cause a close examination to be made of any underwater repair work undertaken since the last survey and, if the manager of a fixed installation has had occasion to take emergency measures to control scour, the

inspection should include the condition of any permanent scour prevention works.

Following the above inspections the surveyor may require that non-destructive tests be carried out to verify, or discount, suspected defects.

Behaviour records of structure and foundations should be examined, where available, and enquiry made into any apparent abnormality.

An assessment should be made of the thickness of marine growth on typical members or areas of the structure. All equipment is to be examined to ensure that it is in accordance with the requirements set out in Parts III and IV of this publication.

If practicable, the second annual survey of a mobile installation should be carried out in sheltered waters; all other annual surveys may be carried out on location when weather conditions permit.

2.3.5 ADDITIONAL SURVEYS

Additional surveys carried out under Section 7 of the Regulations, should be in accordance with the principles laid down for major surveys in so far as the structure may be affected, either locally or as a whole, by alteration, deterioration or damage.

2.3.6 CHRISTMAS TREES AND BLOW-OUT PREVENTERS

The basic design parameters such as maximum working or design pressures and temperatures and fluid or other substances to be handled will be required to be laid down by the owner/operator of the installation and will be his responsibility. The certifying authority will assess all parts with regard to these requirements and the minimum environmental temperature and in accordance with suitable recognized standards.

A maintenance programme for Christmas trees and blow-out preventers should be set up and the owner should obtain agreement from the certifying authority that the maintenance programme with respect to these items ensures that an adequate level of surveys both in extent and frequency is achieved. Such a system should be considered as fulfilling the statutory requirements for annual and major surveys.

2.3.7 SUBSEA WELL AND MANIFOLD PROTECTION

The need for such protection should be assessed by the operator having regard to all possible hazards that may effect the well or manifold and at least include possible damage from anchors, trawl boards and both operational and accidental aspects, including possible iceberg scour. Where it is considered that there is a need to provide protection the operator should define the type and required impact resistance for design purposes. The certifying authority will restrict its assessment to the consideration of the design in accordance with the criteria determined by the operator and the quality control of construction. It is intended that such protection be exempted from the need for annual and major surveys but that a visual survey once every five years or earlier if conditions dictate will be required.

PART II STRUCTURAL REQUIREMENTS Part II Section 1

SECTION 1 DEFINITIONS AND RULINGS

SUMMARY

Section 1 contains a number of technical definitions and rulings that are relevant to Parts II, III and IV.

1.1 AIM OF THE GUIDANCE

The technical requirements set out in the Regulations deal with the broad principles to be followed. The guidance given in Parts II to IV of this document amplifies these principles and refers to standards that will be accepted when consideration is given to an application for a Certificate of Fitness under the Regulations.

1.2 COMPETENT PERSON

A competent person is one who would be recognized as competent by those normally engaged in or responsible for similar activities. If the task is one usually entrusted to persons holding qualifications bestowed by, or recognized by, a professional body, he would be expected to possess such qualifications, and where experience is normally required in addition to qualifications, he would be expected to have had such experience. Similar principles would apply where the individual could be expected to be skilled in a particular trade or other activity.

1.3 RESPONSIBILITY FOR DESIGN, CONSTRUCTION AND SUPERVISION

All aspects of design, construction and supervision should be undertaken by, or under the control of, competent persons.

1.4 CANADIAN STANDARD SPECIFICATIONS AND CODES OF PRACTICE

Subject to the proper exercise of competent engineering judgement and to any qualifications stated in this publication, compliance with any Canadian Standard Specification and/or Code of Practice quoted in Parts II, III and IV will be deemed by the Minister to meet the requirements of the Regulations. Nothing in this section is intended to restrict the right of the owner and his designer to have an installation designed and

constructed to standards equivalent or higher than those set out in this publication.

In some cases a Standard Specification or Code may incorporate alternative clauses and/or may allow certain matters to be agreed between '... supplier and purchaser...'. In such cases the applicable sections should be agreed between the owner and the certifying authority.

1.5 FOREIGN SPECIFICATIONS AND CODES

For brevity reference is, in general, only made to Canadian publications. Other standards and/or codes may be used, subject to the same qualifications as in 1.3, provided the certifying authority is satisfied that an equivalent degree of safety will be achieved. Where no equivalent Canadian publication exists reference may be made to foreign specifications and/or codes in general use in similar circumstances.

Selection from the recommendations in two or more specifications or codes should be permitted only if the selections are in all cases in the direction of increased safety and provided that the result is in every way consistent with sound engineering practice.

1.6 APPENDICES AND REFERENCES

The appendices and references quoted contain data or describe techniques that have special relevance to the sections with which they are associated. They are intended to indicate the minimum technical standards which should be achieved but are not intended to prohibit the use of any equivalent data which may be available. It remains the responsibility of the user to be satisfied on the validity and applicability in any specific case.

1.7 ACCIDENTAL DAMAGE

Under other than extreme loading conditions the usual types of fixed installation can be expected to have a substantial margin of strength and be capable of withstanding considerable structural damage without collapsing (reference 29).

Mobile installations must remain afloat and stable with any two adjacent compartments flooded. (See Section 4.3)

1.8 SAFETY DURING TOW-OUT

The certification scheme does not cover the safety of a fixed installation during transit to the intended location; as it is not then considered to be an established installation. (Part I, Section 1.4, refers).

But during this period the certifying authority must be satisfied that the primary or secondary structures and equipment have at no time been damaged or over-stressed.

1.9 TETHERED BUOYANT INSTALLATIONS

These can be defined as any buoyant installation that is constrained, by mooring lines or other members connected to the sea-bed, to float below its natural level.

The Regulations and the general principles set out in Parts II, III and IV will still apply and owner and certifying authority should, for all new concepts, ensure beyond reasonable doubt the safety of the concept within the framework of the present certification scheme.

1.10 RETROACTION

All current design or construction and all alterations and repairs should be designed and carried out in accordance with the requirements set out in this edition, but existing units and installations in an advanced state of design or construction need comply only as far as reasonably practicable as determined on an individual basis.

SECTION 2 ENVIRONMENTAL CONSIDERATIONS

SUMMARY

This section describes the environmental considerations that should be taken into account during the design and construction of offshore installations for, and in use in relevant waters. It outlines the environmental information which should be obtained and applied.

2.1 ASSESSMENT OF ENVIRONMENTAL FACTORS

2.1.1 INTRODUCTION

A realistic assessment of the environment is fundamental to sound design; only on this basis is it possible to calculate the loads likely to be imposed by natural phenomena and to predict the behaviour of structures under extreme conditions and prolonged exposure. Unlike other loads, however, the occurrence, duration and severity of environmental loads can be predicted only as probabilities, and the data upon which to base probability calculations are not precise and may be limited in scope. Where competent authorities differ the basis of their predictions should be examined critically and appropriately weighted before selecting the data to be used in design.

2.1.2 ENVIRONMENTAL FACTORS

The assessment of environmental factors should quote the sources and authorities consulted, and each report relating to a station for an offshore installation should include information on water depth, sea-bed, wind, waves, currents, sea ice, icebergs, air and sea temperatures, marine growth, snow and ice accretion, and seismic activity.

Normally the latest available authoritative information should be used, but if the assessment is based on data and methods of extrapolation and prediction other than those in current use in official Canadian establishments, sufficient supporting information should be provided to demonstrate their validity and to facilitate comparison with the official data.

2.1.2.1 Water Depths

The water depth should be referenced to a datum which is consistent with all other references to elevations and depths.

The accuracy of the bathymetric survey should be evaluated and account taken of possible errors.

In establishing expected still-water levels and in evaluating design details, allowance should be made for tidal variations, including asymmetry of tidal height, particularly in depths of less than 20 metres, and of increases or decreases in water depth due to wind and pressure-induced storm surges. In depths greater than 20 metres it is usually sufficiently accurate to add half the appropriate value of tidal range to the surge height to obtain the maximum still-water height above mean water level.

2.1.2.2 Wind

Wind is of importance from two main aspects: (a) because a structure must be designed to withstand the forces exerted on it by the wind itself (reference 1), and (b) because the heights of waves (and hence the wave forces to be resisted) depend inter alia upon the speed and direction of the wind over some preceding period of time (reference 2).

Wind conditions to be considered for design may be described either by statistical or deterministic methods. For extreme conditions these should be for an average recurrence period of at least 100 years. The validity of the procedures used is to be documented.

Wind speed statistics are to be used as the basis for a description of the long-term wind conditions. The long-term distributions should preferably be based on statistical data for the same averaging periods of wind speed which are used for the determination of loads. If data is available for other averaging periods only, such data may be converted by applying appropriate gust factors.

Extreme values of gust and sustained wind speeds are to be expressed in terms of most probable largest values with their corresponding recurrence periods.

The wind speed at a height 10 meters above the still water level is to be used as a reference for both sustained and gust wind speeds.

(a) Estimating Wind Speeds

There are some serious difficulties in estimating probable maximum wind speeds over the sea, the most important being a lack of suitable measurements. At sea most observations of wind speed are made at fixed hours, usually 0, 6, 12, 18 h GMT from moving ships, without the aid of instruments. Thus for any particular offshore site the available observations would normally consist of estimated wind speeds made at irregular intervals by a variety of observers from different ships, so that the chances of getting accurate observations of the highest speeds that have actually occurred there are very poor, particularly when it is remembered that vessels tend to take shelter when winds exceed about 20 metres per second. The exceptions to this are the observations made from ocean weather ships (OWS) which provide regular wind observations from more or less fixed positions, based on well-exposed and accurate anemometers. Even so, continuous records are not available, the observations being spot measurements, made either hourly or three-hourly.

Because of these difficulties it is considered that estimates can be made of the probable maximum wind speeds over offshore areas largely based on records from coastal stations, supplemented by observations, as they may become available, from offshore installations themselves.

(b) Design Wind Speeds for Wind Loading

Wind loads and local pressures are to be calculated in accordance with a recognized standard.

Data obtained from reliable and adequate model tests are recommended for the determination of pressures and resulting loads on structures of complex shape.

Wind loads calculated on the basis of 1 minute sustained wind speed may be used in combination with maximum wave loads, where the wind load may be treated as a static load. For structures which are sensitive to dynamic wind loading (eg high masts and stacks, flarebooms, compliant platforms) the dynamic effect of the wind should be included in the analysis. If wind loads due to the three seconds gust wind acting alone are more unfavourable than the wind loads due to the one minute sustained wind combined with wave loads the three second gust wind acting alone should be considered. In the case of wide structures or structural elements, the averaging period of the wind speed used should be related to the dimensions of the structure.

(c) Design Wind Speeds for Prediction of Extreme Waves

The wind speed intended for use in predicting probable maximum wave heights should be averaged over a longer period than for wind loading, e.g. one hour, and should be accompanied by factors allowing estimates of probable maximum speeds averaged over longer periods (up to 24 hours) to be made.

Wind inputs for wave hindcasting models may also be derived by accepted hindcasting techniques from pressure field data.

(d) Seasonal Variations of Extreme Wind Speeds

Questions may arise regarding the possible restricted use of a mobile installation in an area having probable extreme wind speeds in excess of its design wind speeds. Such use would be limited to those months of the year when the probable extreme wind speeds are not likely to reach the design speeds. In such cases, wind statistics could be developed by including data not only from the proposed operating period but for at least one month before and after this period.

(Note: Other wind speeds may also need to be taken into account, e.g. in calculating reserve stability.)

(e) Frequency of Wind Speed and Direction

When planning and carrying out various phases of construction and considering operating arrangements, information on the frequency of wind speeds and directions will be required.

2.1.2.3 Waves

(a) General

One of the predominant contributions to environmental forces on offshore installations comes from water waves. The information relative to waves used in the design should include heights, periods, directions and water depths; it should be clear what assumptions have been made, particularly those relating to limiting conditions. It should be borne in mind that the waves imposing maximum loads on the structure as a whole may be different from those having the most severe effect on structural elements. It should also be noted that it may be lesser, more frequent waves which govern fatigue life.

Wave conditions may be described either by statistical or deterministic methods. The valadity of the procedures used is to be documented.

The selection of suitable parameters for design purposes is in both cases to be based on the use of wave statistics or accepted hindcasting techniques.

(b) Information Needed for Design Purposes

(i) The 100-year design wave. The minimum design wave considered should be that having an average recurrence period of not less than 100 years. For an installation standing on the sea-bed, this wave may be the one which will develop maximum loads on the structure as a whole and on some, or all, of its principal elements and the maximum crest level of this wave will be needed to calculate the clearance height for the superstructure. The behaviour of a floating installation subject to the 100-year wave also has to be examined even though maximum stresses in the structure may arise under other conditions.

(ii) The wave, or group of waves, giving rise to maximum loads on the structure as a whole and on its principal elements, taking into account both wave height and wave period.

Analysis may show that the maximum loads may be developed in response to a wave, or group of waves, other than the 100-year wave described above; this is likely to apply particularly to floating installations but may also be significant for sea-bed supported installations and their principal elements. In the case of a floating installation account should be taken of the wave, if other than (i) above, that will give minimum clearance from any part of the installation not designed to withstand wave impact.

(iii) probable distribution of wave encounters. An estimate of the probable distribution of wave encounters that the installation is likely to experience during its service life is essential for assessing whether significant fatigue effects will develop in its structural elements.

(iv) Wave energy spectra. Alternative methods of carrying out load, stress and fatigue analyses may be based on the use of wave energy spectra.

The representation of the sea surface by a linear superposition of sinusoidal waves results in the concept of the wave spectrum (references 3-6). The distribution of the energy or squared amplitude of the elementary sinusoidal components with frequency is called the one-dimensional wave spectrum. This concept can be extended to include wave direction, that is, the directional wave spectrum.

Analytic forms for the one-dimensional spectrum of waves are available for certain idealized and limiting deep-water conditions.

Analytical wave spectra are to reflect the width and shape of typical spectra for the site considered. For open sea areas, the Pierson-Moskowitz type of spectrum will normally apply (reference 7). Other spectrum formulations will be considered under special circumstances (references 8-17).

In general, measured wave spectra show a wide variety of shapes due mainly to the space and time variability of wind fields over the oceans. In many oceanic situations measured wave spectra are bi-modal with contributions due to swell superposed on a locally generated wind sea. (In shallow water and near the coasts bottom and topographic features have major influences in modifying the offshore, deep-water spectra.)

The short-crestedness of waves in a seaway, i.e., the angular distribution of wave energy, may be taken into account. If detailed field measurements are not available a cosine to the fourth power distribution will normally be accepted.

Measured wave spectra are only available at a few locations in the oceans on a climatological scale (references 18-20). For design purposes it may therefore be necessary to parametrize these spectra in terms of measured or visually observed wave statistics such as significant wave height and average period. Hindcast wave spectra generated by numerical wave models which use wind field information may also be an important source of data.

(c) Available Data

There is a limited amount of measured wave information available on wave climate around the coast of Newfoundland and Labrador. This information is available from the Marine Environmental Data Service. This information can often be accepted as being representative of wave conditions over a typical year, for a particular location.

(d) Extreme Wave Heights

There are two main methods which can be used to estimate severe conditions: one is to use calculations of extreme wind speeds and to apply them to a wave forecasting technique (reference 2). The other is to statistically extrapolate a series of measured wave data.

Extreme values of wave heights are to be expressed in terms

of most probable largest values with their corresponding recurrence periods.

Long-term predictions are to be based on recognized techniques.

In design using deterministic procedures based on regular wave considerations, the wave is to be described by the parameters wave height and period. The design wave formulation used is to be valid for the problem considered.

The design waves or sea states are to be those resulting in the most unfavourable effects on the structure or structural part considered, taking into account the shape and size of the structure, water depth, etc. Consideration is to be given to the probability of occurrence of these design waves or sea states.

A range of realistic periods should be established for each wave height and the value least favourable to the structure should be used in design.

The wave period is to be specified in each case of application. It may be necessary to investigate the wave loads for a range of wave periods in order to ensure a sufficiently accurate determination of the maximum response.

(e) Modification of Waves Due to Topography

Account must be taken for sea-bed topography, refraction or reflection from surrounding land masses. For any selected site detailed studies should be undertaken covering a large area surrounding the site so that due account may be taken of any features likely to focus, or attenuate, waves, even from considerable distances, or severely confused seas which may occur at the site. Studies based on calculation should be supplemented to the maximum extent practicable by site observations and with routine observations.

2.1.2.4 Currents

There is a lack of both adequate measured data and theoretical understanding of the near-surface currents which are of greatest importance in connection with calculation of forces on offshore structures. Data sequences of adequate length are only just beginning to be obtained. Techniques of analysis designed to determine parameters appropriate to engineering design are being investigated.

The total current to be taken into account should include

tidal currents, currents associated with surges, wind-generated currents and any currents due to other causes. Unless a detailed study of current directions has been made, it should be presumed that currents may run in any direction. It should be noted that there is some evidence that the strength of currents, particularly sub-surface currents, have been under-estimated in the past.

2.1.2.4.1 Currents and Wave Steepness

When a wave train moves into a current field, the wave height and propagation speed are changed in compliance with certain conditions of continuity of energy flux and conservation of the number of waves; this can lead in some cases to very considerable increases in the height and steepness of waves. Reference 21 discusses this problem and its implications and illustrates the case of waves travelling into a deep vertically-uniform current. An extreme magnification is predicted for the critical case of waves opposed by a current whose speed is a quarter of the wave phase speed, when the wave velocity is reduced to zero and the energy is trapped.

Directional distortions of a wave system due to gradients of current velocity can also lead to high local concentrations of energy and corresponding increase of wave steepness. The most documented effects on shipping of the possible modification of waves by current are to be found in the Agulhas Current off South Africa, where the current opposes swells from the Southern Ocean (reference 22).

2.1.2.4.2 Currents and Structure Loads

In the case of jackets and other space frame structures, where wave-induced loads involve substantial components of drag force which are proportional to the square of the ambient velocity, the effect of an added current on the load can be very large. This point is explained, for example, in reference 23 where it is noted that according to the standard Morison formula for the drag force in waves, a one-knot current superimposed on a 10 meter high 10-second period deep-water wave will increase the drag near the surface by about 40 percent.

2.1.2.5 Marine Growth

Generally, areas where the growth is dominated by mussels or various types of algae, the growth will be most extensively close to the surface, as they do not go deeper than 30-40 meters below mean sea level. For the areas where sea

anemones, hydroids, sea squids, etc. are dominanting, the growth will be most extensive near the bottom.

Extensive marine growth will increase the sectional area of a member and alter its surface characteristics, both changes tending to increase resistance to waves and currents and to increase the loads applied to the structure. The increased diameter taken into account in the design should be stated in the Operations Manual and the stated thickness should not be exceeded.

2.1.2.6 Air and Sea Temperatures

(a) Air Temperatures

Percentile values of air temperature, temperatures below which 1, 5, 50, 95 and 99 percent of the observations lie in the coldest and warmest months (usually February and August) in a number of areas around our coasts are given in reference 25 and typically display a fair weather bias.

Little data is available for the more northern areas, but extremely low temperatures can be expected near the ice edge. Temperatures as low as $-22^{o}C$ have been reported as far south as the Straits of Belle Isle area (reference 27).

(b) Sea Surface Temperatures

A summary of the monthly variation in sea-surface temperature is given in references 26 and 28.

Minimum sea-surface temperatures of $-2^{o}C$ can extend over the entire eastern offshore area with the southward advance of sea ice in winter. Maximum surface temperatures of approximately $20^{o}C$ can be reached on the Grand Banks in the warmest month of August.

(c) Mid-Water and Sea-Bed Temperatures

Very unusual temperature profiles exist in many of the Newfoundland and Labrador offshore regions because of the complex current regimes present. Thus any assumptions regarding sub-surface temperature should be made with extreme caution.

However, both minimum and maximum sea temperatures occur at the surface and these may be used as design maximum or minimum at all depths.

2.1.2.7 Sea Ice and Icebergs

(a) General

There are zonal and seasonal variations in ice activity in waters related to these guidance notes. Ice begins to form on the northern Labrador coast in early December. The extent of ice advances southward as winter progresses. Second and multi-year ice move into the region from the Arctic and rafting and ridging of first, second, and multi-year are present.

The presence of ice considerably influences offshore installations planning, design, and operations. The planning considerations should relate to the low temperatures and the presence of ice together with the other offshore environmental conditions prevalent in more temperate areas.

(b) Environmental Considerations

The design environmental conditions should be those having an average recurrence period of not less than 100 years. Information on the following is necessary for the design of an installation.

(i) Sea Ice

1. Shorefast ice. Information on the areal extent and thicknesses for certain coastal locations is available from charts published by Ice Forecasting Central, Atmospheric Environment Service, Federal Department of Environment. Thicknesses should be determined when considering thermal and vertical forces.

2. Sheet ice. Sheet ice should be considered in the design of offshore structures in protected bays and lagoons. Thickness and movement should be determined when considering ice forces on structures.

3. Rafted ice. Rafted ice may be two or more ice sheets thic[illegible]nd tends to consolidate rapidly due to the absence of pore water. Rafted ice thicknesses and distributions should be determined when considering ice override potential and ice forces.

4. First-year ridges. The sail of the ridges is usually composed of loosely consolidated blocks. The lower portion of the keel could be consolidated and the size, movement rate, and thermal properties should be determined in considering ice forces on structures.

5. First-year floes. Information on the extent and concentration of the floes is available from charts published by Ice Forecasting Central. Thicknesses and movement rates should also be determined in considering ice forces on structures.

6. Rubble features. Areal and thickness distributions, movement rates, and consolidation properties for rubble fields should be considered as potential design parameters.

7. Multi-year floes. These floes which may contain embedded ridges or rubble piles, have weathered, rounded, consolidated sails and relatively solid keels. The sizes, distributions, and movement rates should be considered as design parameters.

(ii) Sea ice - physical properties. Sea ice is a crystaline material, its properties will depend on the size and orientation of it's crystals, and its salinity, temperature, density, and impurities.

(iii) Sea ice - mechanical properties. The mechanical properties of sea ice are dependent on many factors, including its crystallographic structure, temperature, brine volume, and strain rate. Depending upon the level and rate of loading, the ice behaves elastically, creeps or fractures. Information on various strengths, flow and fracture characteristics would assist in estimating loads under various conditions.

(iv) Icebergs

Icebergs due to their large mass can impose considerable loads on the structure. Various characteristics of the iceberg affect the magnitude of the loads and iceberg towing, or blasting and abandonment procedures.

1. Distribution. Information on the distribution of icebergs is available from Ice Forecasting Central. The probability of iceberg impact would depend on the number of icebergs.

2. Size. The mass of the iceberg is a major factor that governs the impact load. The size distribution of icebergs would also govern the detection and towing procedures.

3. Shape. Icebergs are classified by the above water size and shapes. The shape governs the stability of the iceberg and gives information on the degree of deterioration of the iceberg. It would also govern the type of failure at impact and the load on the structure.

4. Draft. Information on iceberg drafts are necessary to evaluate scour potential and clearence for seabed facilities. It would also govern the design of protection schemes.

5. Velocity. The drift and velocity of icebergs would be necessary to determine the loads, type of detection devices and abandonment procedures.

6. Stability. The roll of icebergs could alter the draft and govern towing procedures. Calving of icebergs also affect the stability of the iceberg.

7. Drag coefficients. These are needed for iceberg drift prediction.

8. Properties. Very little information is available on the properties of icebergs. Information on the following is needed for the calculation of impact loads and for iceberg management.

- Crystal structure. Icebergs calve from glaciers formed of ice originating from snow.
- Physical characterists: Cracks, bubbles, etc.
- Density. Air bubbles are entrapped in the ice which lower the density and strength.
- Temperature.
- Compressive strength. The pressure on the structure is limited to the strength of the ice and is a function of the state of stress (multiaxial).
- Tensile strength. This would govern the calving of icebergs and breakup at impact.
- Elastic properties: Elastic modulus, Poisson's ratio, and yield strength.
- Fracture toughness.
- Compressibility, velocity of wave propagation, and attennuation of stress waves govern the explosive deterioration of an iceberg.

(v) Iceberg and ice ridges scour.

Icebergs and ice ridges scour the ocean floor in the process of grounding. The frequency of occurrences, distribution, depth, orientation, and width of the ice scours should be known. The seabottom systems should be buried below the maximum depths of scour and the effect of forces transmitted due to scouring/grounding should be included in the design, or should be designed to be expendable, but without adverse environmental effects.

2.1.2.8 Snow and Ice Accretion

Under severe conditions, snow and ice may accumulate to such an extent as to have a significant effect on the stability of floating units and the wind resistance of exposed structural elements will be increased by the growth of ice.

An estimate should be made of the extent to which snow and ice may accumulate under any particular condition and of its possible effect on the structure concerned.

(a) Snow

The probable maximum depth of snow in a single fall should be determined. The snow may be assumed to have a density of about $100kg/m^3$ initially. It would be most likely to adhere and accumulate on horizontal and windward facing surfaces when the air temperature is not far below 0^oC.

(b) Icing from Sea Spray

Icing from sea spray can occur when the air temperature is below about -2^oC and the wind is sufficiently strong for spray to be blown from the wave crests or to be formed and thrown upwards as waves break against the structure (reference 24). Less ice would collect near sea level because they would be frequently bathed in normally somewhat warmer sea water. The upper parts of the structure would also collect less than the lower section because amounts of spray decrease with height. The average density of ice formed from sea spray is about $850kg/m^3$.

(c) Icing from Freezing Rain or Drizzle

Icing from freezing rain or drizzle is known as glazed frost or glace icing. Glazed frost is a dense, clear deposit of ice with a density of about $900kg/m^3$, which forms when drops of rain or drizzle cooled below 0^oC fall on to an exposed surface, freezing on impact. It is a phenomenon occurring when persistent cold winds are overlain by a shallow layer of warmer

air into which snow is falling. The snowflakes are melted in the warm layer and the resulting drops are then supercooled as they fall through the freezing surface layer.

(d) Icing from Freezing Fog or Cloud Droplets

Overland rime icing from cloud droplets seems only to be a serious problem for structures on hills at heights of more than 180 metres above sea level. This is because heavy rime icing requires prolonged exposure to supercooled cloud or fog droplets together with moderate to strong winds. It is difficult to envisage such conditions occurring at lower levels than this over the sea for a sufficiently long period to give appreciable deposits.

(e) Snow and Icing Combinations

Possible simultaneous occurrence of snow and of ice due to the above causes could occur in the following combinations:

(i) sea spray plus glared frost;
(ii) sea spray plus snow;
(iii) sea spray plus rime;
(iv) snow plus rime.

Sea spray plus glared frost is likely to produce the largest accumulations and should be allowed for by assuming a maximum thickness of glazed ice on upward and windward facing surfaces up to 15 meters above sea level and decreasing upwards.

2.1.2.9 Seismicity

The effects of earthquakes are to be considered for structures to be located in areas that are determined to be seismically active.

An area is considered seismically active on the basis of previous record of earthquake activity, both in frequency of occurrence and in magnitude.

For areas where detailed information on seismic activity is available, the seismicity of the area may be determined from such information.

For areas where detailed information on seismic activity is not generally available the seismicity is to be determined on the basis of detailed investigations, including a study of the geological history and the seismic events of the region.

If the area is determined to be seismically active and the structure will be affected by earthquake effects, an evaluation

is to be made of the regional and local geology in order to determine the location and alignment of faults, epicentral and focal distances, the source mechanism for energy release and the source of site attenuation characteristics, taking into account the local soil conditions as far as these may affect the ground earthquake and the maximum credible earthquake.

2.1.2.10 Official Sources of Information

Enquiries for any additional or more recent authoritative environmental data may be made to the following sources:

Atmospheric Environment Service
Atlantic Region,
1496 Bedford Highway,
Bedford, Nova Scotia B4A 1E5

Bedford Institute of Oceanography
P.O. Box 1006,
Dartmouth, Nova Scotia B2A 4A2

Marine Environment Data Service
Department of Fisheries and Oceans
Bay 234, City Centre, 880 Wellington Street,
Ottawa, Ontario

2.2 APPLICATION OF ENVIRONMENTAL INFORMATION

2.2.1 VALUES TO BE TAKEN INTO ACCOUNT DURING DESIGN

The most unfavourable combinations, position and direction of the forces have to be taken into account in the design. The combination and severity of environmental factors to be taken into account should be in a manner consistent with the probability of their simultaneous occurrence supported by sufficient statistics. Earthquakes normally need not be considered to act simultaneously with other environmental loads.

Any other combination of environmental factors which may cause greater stresses either in the structure as a whole or in any element of the primary structure, should be taken into account unless such conditions are likely to occur less frequently than once, on average, in any period of 100 years.

For mobile installations due consideration should be given for iceberg towing support and limiting environmental conditions for which they would be on location. Care should be taken that the associated procedures and movement are possible for all probable combinations of the other environmental factors.

Mobile units that operate during the ice season but are intended to move away from approaching ice (icebergs) should be designed to withstand the impact of the largest piece of ice that could fail to be detected by the iceberg detection system on that unit.

2.2.2 EXTREME ENVIRONMENTAL CONDITIONS

(a) For each mobile unit, the limiting operating environmental conditions will need to be determined, including water depth, wind, wave, current and ice characteristics and, additionally for bottom-supported units, sea-bed characteristics.

(b) In defining the limiting environmental conditions for a drilling ship which is designed in accordance with Canada Steamship Act, account should be taken of zones and seasonal areas as defined in the International Convention on Load Lines 1966.

(c) If a mobile unit is only to be used during a season or seasons when the environmental conditions are likely to be less severe than those for the whole year, due allowance may be made for this in deciding upon the limiting conditions.

(d) For fixed installations and mobile units intended to be on station for an extended period of time, an assessment should be made by a competent person of the environmental conditions expected at the station, which will form the limiting environmental conditions to be taken into account in determining the suitability of the unit for the particular site. It should be shown that these conditions are consistent with those appropriate to the area and that account has been taken of local conditions that may modify wave characteristics.

2.3 OPERATIONS MANUAL: ENVIRONMENTAL CRITERIA

The Operations Manual should contain relevant information as to environmental considerations and their effect on the installation, for the use of the installation manager. This should cover the way in which extreme environmental conditions may effect the installation generally, and the effect of wind, sea states and the accumulation of marine growth, snow and ice on the stability and fitness. Information should be included on the particular subject, guidance when action may be needed and instructions on the action to be taken, all relative to the various modes of use, whether on station, or for mobile units, the placing on and removal from station, and movement between stations.

2.3.1 FIXED INSTALLATIONS

The manual should contain the design compliance limits as the fixed installation remains in place during its term of operation. It should also contain procedures to be followed in the case of a severe environmental occurrence (iceberg impact, earthquake, etc.) and details regarding key points of the structure to be surveyed and evaluation of monitored response of the structure to ascertain that no damage has occurred. This should also cover procedures for establishing the installation such that design compliance is available and the limiting environmental conditions at which the procedures can be followed.

2.3.2 MOBILE INSTALLATIONS

The manual should contain limiting environmental and response parameters for various operations. It should also contain information on detection devices and forecast models and procedures to suspend operations or move away from location under prevalent limiting environmental conditions. It should also contain procedures to be followed in the case of a severe environmental occurrence (ice or bergybit impact etc.) and details regarding key points of the structure to be surveyed and evaluation of monitored response of the structure to ascertain that no damage has occurred. Procedures to be followed for iceberg towing support (mass, size, etc.) should also be included.

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SECTION 3 FOUNDATIONS AND SITE INVESTIGATIONS

SUMMARY

This section gives guidance on foundation design for installations intended to be supported, either permanently or from time to time on the sea-bed. It also covers the site investigation that should be made and the steps that should be taken to select the soil properties to be used in foundation design.

More often than not the type of foundation to be adopted is decided in the light of existing knowledge of the area and the subsequent site investigation is concentrated on aspects having specific relevance to that type of structure. Foundation design is therefore considered first but it should be clearly understood that design and investigation are interdependent and must be considered together.

3.1 FOUNDATIONS

3.1.1 GENERAL

Most foundations for offshore installations can be classified as either:
(a) piled; or
(b) gravity, including mobile units supported by the sea-bed.

In piled foundations, the piles usually take the form of open-ended steel tubes installed either through the legs of the installation or through sleeves attached to the jacket structure, at or about sea-bed level, and driven to depths below the sea-bed commensurate with the applied loads and the strength of the sea-bed material.

Gravity foundations rely on the weight of the installation to provide stability against the horizontal forces and overturning moments produced by waves, currents, wind, ice and also berthing and other loads with horizontal components.

The sea-bed elements of a restrained buoyant installation may take the form either of slabs or blocks relying on gravity for stability, or of piles, screw piles, grouted piles, self-burying anchors or other devices that mobilize the resistivity weight of the sea-bed either by friction or by mechanical means.

3.1.2 CODES OF PRACTICE

The Canadian Foundation Engineering Manual (CFEM) published by the Canadian Geotechnical Society relates to the foundations of land-based structures but many of the contained recommendations are equally applicable to offshore structures. Other codes have been published or are in preparation, dealing specifically with offshore installations, but before applying the recommendations in any code their applicability in specific offshore circumstances should be critically examined.

3.1.3 DESIGN

The performance of the structure and the sea-bed should be considered together.

Foundations should be designed so that under the most severe loading conditions the imposed load does not approach the ultimate bearing capacity of the sea-bed and the underlying ground. Further, the allowable bearing pressure should be selected to ensure that expected long-term settlement is acceptable and will cause no damage either to the installation or to risers, pipelines or other connections.

The factor of safety to be used depends upon the degree of accuracy that can be ascribed to the soil parameters actually used in design. The design soil parameters should be assessed on the basis of the results from properly planned and executed in-situ and laboratory tests, with due consideration to relevant geological information and available experience from previous installations in the area (reference 1). It is recommended that the factor of safety against bearing capacity failure and against sliding be not less than 1.5, used in conjunction with the design soil parameters and the most severe loading conditions, and not less than 2 under conditions likely to occur not more than once, on average, each month.

3.1.4 PILED FOUNDATIONS

Maximum compression, tensile and bending loads on each pile, resulting from all possible loading conditions, including lateral loading, should be considered. The piles should be designed to carry the axial, bending and shear stresses resulting from these loads (reference 2). Consideration should be given to the possibility that long-term cyclic loading, due to lateral and axial load reversals, may cause plastic deformations of soil, especially along the upper portion of the pile. Lateral loading aspects present special problems (references 5 and 6).

During pile-driving, dynamic measurements to monitor and verify the hammer performance and the energy input to the pile should be made. These measurements, along with continuous records of the number of blows per unit of penetration, provide a useful correlation between the pile driving resistance and the soil strength profile. Through further analysis, these data may be used for prediction of the static capacity of the pile at the time of driving. Any backcalculation of pile capacity from pile driving data should, however, give due consideration to the effects of pore pressure dissipation in the soil, after driving.

In clays, dissipation of pore pressure after driving can result in a considerable increase of load-carrying capacity due to the increase in horizontal effective stress (reference 3). In some silts and sands, a reduction of pore pressure caused by dilation, as the pile sets up shear stresses in the soil, may temporarily increase the resistance to penetration. A return to the normal pore pressure values after driving may reduce this resistance, which should be checked by re-driving the piles. If the resistance proves to have been reduced, and is not attributable to lifting of the point as described below, consideration should be given to reducing the pile loading, e.g. by driving additional piles. With end-bearing piles there is a possibility that driving subsequent piles may raise adjacent piles. A check should be made of the levels of the pile heads and, if necessary, allowance made for the loss of end bearing.

The load that can be supported by a group of piles may be less than the multiple of the single pile load-bearing capacity and the number of piles in the group. In sand, driving a group of piles may cause permanent consolidation, with a significant increase in the bearing capacity of the group, but in clays a decrease can usually be expected. Where, as is frequently the case, pile groups penetrate successive sand and clay layers, the net effect should be calculated as accurately as possible (reference 4).

3.1.5 GRAVITY FOUNDATIONS

A gravity installation will normally apply loads over a substantial area of the sea-bed and the distribution of the total load should not be assumed to be uniform unless there are specific grounds for so doing (reference 7). Actual load distribution under any given loading condition will depend upon soil characteristics and response down to depths at least comparable to the dimentions of the foundation. Foundation pressures and stress distribution will also be affected by surface irregularities in the sea-bed. An accurate

topographical survey of the sea-bed is necessary and it is desirable, in addition, to make a visual inspection using manned or remotely controlled equipment. The effect of pore water pressure distribution beneath gravity bases during the passage of a wave should be considered. This could be crucial to shallow water structures.

Where the sea-bed is irregular the design should allow for such changes in foundation pressure and stress distribution as may result from the installation being located at any point within planned tolerance limits.

Reasonable estimates for skirt penetration forces can be derived from cone penetration test results and values can be estimated from the shear strength parameters for clays.

The detailed design of a gravity base may have to incorporate provision for grouting to be carried out during installation, both to control levels and to ensure predetermined load distribution.

3.1.5.1 Bottom Supported Mobile Units

The suitability of a certain mobile unit for a given location is dependent upon, in addition to factors mentioned above, the depth of leg penetration, stability considerations and necessary preload.

The preload capability is normally given in terms of the maximum vertical load which can be applied by ballasting. It should be emphasized that such pure vertical preloading is insufficient to verify that the unit is safe during storm conditions since the horizontal load component reduces the bearing capacity considerably. Ultimately this may lead to punch through failure or general bearing capacity failure.

3.1.6 SETTLEMENT

The problem of settlement is generally associated more with gravity than with piled foundations. The maximum overall settlement should not reduce the distance between the maximum wave crest and the underside of the platform below the clearance specified in 5.2.2. Differential settlements should not be greater than the structure can safely withstand and, to avoid damage to the risers, special provision may be required to allow for differential movement between the structure and the risers.

Settlement may be caused by consolidation and displacement of the soil and by the bedding-down effects of cyclic loading (references 8 and 9).

3.1.7 CYCLIC LOADING

Cylic loading can lead to a progressive increase of pore pressure within the bottom soil, particularly in loose sediments, if the increase of pore pressure is large, liquefaction can occur, with an almost complete loss of shear strength. This is most likely if cyclic loading is so rapid that excess pore pressures cannot dissipate. On the other hand, if the liquefaction threshold is not reached, subsequent pore-pressure dissipation will increase settlements, perhaps with some accompanying gain in strength because of a reduction in voids ratio.

Analysis of the behaviour of a gravity foundation under cyclic loading is a very complex matter requiring sophisticated analytical techniques and relevant soil data for both static and cyclic loading conditions. In these circumstances consideration should be given to the use of model tests such as centrifugal model tests. Both centrifugal model tests and analytical studies require that the soil sample properties approximate closely to the actual site conditions.

If a gravity structure rocks under cyclic loading a pumping action may become established, sea water being alternately drawn in and forced out below the base. This is a highly dangerous situation, fine material being carried out by the high-pressure flow with consequent loss of positive bearing pressure and progressive deterioration. A skirt penetrating the sea-bed will reduce this hazard.

3.1.8 SEA-BED MOVEMENT

Movement of materials on the sea-bed can be caused by currents, wave action, or earthquakes and by scouring icebergs or seepage of gas emanating from below the sea-bed. If the site investigations indicate that there is a possibility of sea-bed movement due to any of these causes, then the effect on the foundations should be considered. Any changes in sea-bed currents which may result from the presence of the structure should also be taken into account. The effect of scour may be alleviated by the addition of a skirt or apron to the foundation, or by the placing of materials around the foundation after the structure has been installed. Proposed methods of alleviating scour and other sea-bed movement should be agreed with the certifying authority.

Special precautions must be taken when carrying out drilling operations below or adjacent to a gravity installation. Cases have occurred where significant quantities of sea-bed material have been displaced or removed either by

'hydraulic fracturing of the seabed' due to the drilling fluid being under excess pressure, or to caving below the tube, fine material being brought up by the circulating drilling fluid.

3.1.9 OPERATIONS MANUAL - FOUNDATION CHARACTERISTICS

For appropriate installations, the Operations Manual should contain relevant particulars of the characteristics of the foundations and, where relevant, their suitability for different sea-bed conditions, bottom penetration aspects and limiting values for scour and other sea-bed movements. Guidance should be given when action may be needed, and instructions on the action to be taken, all relative to the various modes of use, on station, and being placed on or removed from station.

3.1.9.1 Fixed Installations

The manual should contain the procedures to be followed in the case of a severe environmental occurrence (iceberg impact, earthquake, etc.) and details regarding foundation and seabed surveys and limiting values for the foundation and seabed movements.

3.1.9.2 Mobile Installations

The manual should contain details regarding seabed surveys to be undertaken prior to any reconnect operations to ensure that no damage has occurred during or after disconnect.

3.2 SITE INVESTIGATION

Information on site conditions can, and should, be built up by combining data from all available sources. Reference may be made to Chapter 4 of Section I of the CFEM.

3.2.1 BACKGROUND GEOLOGY

Information on the geology of the site may be be obtained from adjacent sites, from general surveys such as those made by Government institutions and from published literature. This information will help in planning the detailed site investigation and in proving an assurance that the findings of the site investigation are consistent with the geological conditions.

3.2.2 GEOPHYSICAL SURVEY

High resolution shallow penetration seismic methods may reveal strata thicknesses and variations in their thicknesses over the general area of the site. When these techniques are used they must be specifically designed to show up the soil properties. Therefore, seismic surveys should be interpreted in the light of one or more boreholes in the survey area, or appropriate shallow surveys obtained with tethered devices such as drop covers, grab samplers or vibra covers.

3.2.3 TOPOGRAPHICAL SURVEY

Knowledge of the configuration of the sea-bed is required to identify potentially unstable slopes, shapes which might affect scour behaviour, etc. At the actual site for the structure a more detailed survey is required to establish surface irregularities, average slopes across the site, presence of rock or man-made hazards or obstructions. A submersible or other means such as side-scan sonar will be valuable in observing the condition of the sea-bed. The survey should show any seepage of gas from the sea-bed and the presence of pock marks or small craters. Measurements should be made of current flow near the sea-bed.

3.2.4 SUBSOIL INVESTIGATION

This involves the use of in-situ testing, boring and sampling and should be carried out at the site finally chosen for the structure. The actual extent and complexity of the investigation depends very much on the consistency and uniformity of the foundation deposits and it should be tailored to the design methods used. Unnecessarily conservative designs may be avoided by making the investigation sufficiently extensive and thorough. The objective of the investigation is properly to describe the soil profile, including soil structure and deformation parameters for the soil.

3.2.5 IN-SITU TESTS

The results from all forms of in-situ tests require careful interpretation (refernce 1). Values obtained will be influenced by the type of apparatus, the way it is used and the type of soil being tested. Rates of strain, which may be difficult to control, can be very important.

In every case the operating principles of the equipment, their potentials and their disadvantates, should be evaluated

and taken into account in assessing the accuracy and validity of the results obtained.

3.2.6 BORINGS

Boreholes should be made at the site for each structure so that samples of the various strata may be inspected to confirm the soil types. The number and depth of boreholes will depend on the variation of soil type, size and design of foundation and the amount of reliable information already obtained from other investigations made on or near the site by other methods. Guidance is given in the CFEM, section on site investigations.

It is important to know accurately the positions of all boreholes.

3.2.7 SAMPLING

A sufficient number of samples should be taken to ensure that a reasonably continuous profile, composite if necessary, can be obtained from the subsoils encountered. Samples should be as undisturbed as possible. The soil will be disturbed if boring allows excessive stress relief; this can be reduced by the use of drilling mud of sufficient density. The soil will also be disturbed by a sampling tube with an incorrect shape of cutting shoe and/or an excessively large area ratio. The existing stress and stress ratio in the soil will be altered by sampling and this may also seriously affect soil properties measured by subsequent tests made on samples.

A reasonable number of samples should be extruded on board the survey vessel to ensure acceptable quality and sample recovery. The extruded samples should be tested immediately or sealed for later testing.

The record of the soil profile can be enhanced by a series of colour photographs of undisturbed samples that have been carefully cleaned and split in half without smearing the surface to reveal the colour and texture of the soil strata. Samples to be stored should be kept in their sampling tubes, in a vertical position preferably under an applied stress to minimise swelling and change of structure. Spring-loaded pistons can be fitted to either end of the sample tube and the springs compressed to produce an axial stress in the sample equal to the overburden to the depth from which it was taken. All samples should be stored in a refrigerator at 4 to 5^{o}C and brought to shore at earliest convenience.

3.2.8 SOIL DESCRIPTION

Each sample that is extruded from its sampling tube and split should be described to record its consistency and structure so that an interpretation of its geological history may be made. Special attention should be paid to structural details such as silt and sand partings, fissures, etc., which though easily overlooked, could play an important role in the behaviour of the soil mass.

Samples that are tested should also be split and carefully examined after the test is complete to add to the overall description of the soil profile.

3.2.9 LABORATORY TESTING

The type of laboratory tests required will depend on the type of foundation and the design method as well as the soil type. In general sufficient classification tests should be carried out to establish the soil type in soil mechanics terms. Deformation and ultimate strength tests should be made to establish soil parameters with the degree of accuracy required by the design. These tests are usually made with a triaxial apparatus which can be used to consolidate the sample under a wide range of stress ratios and cause the stresses to follow a wide variety of paths and measure sample deformation, volume change and/or pore pressure alterations. Oedometers, shear apparatus and other special laboratory equipment may also be used to measure soil properties, depending on the design requirements.

Special tests to measure the deformation properties of soil samples under the action of cyclic loading having been described by Brown, Lashine and Hyde (1975), Moussa (1975) and the Norweigan Geotechnical Institute (1975) (references 10-12). These tests should be made where there is any doubt about the possible behaviour of the soil.

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SECTION 4 PRIMARY STRUCTURE

SUMMARY

Primary structure means all those components of an offshore installation, the failure of which would seriously endanger the safety of the installation.

This section includes information on loads and their assessment; on design criteria for steel and concrete structures and for floating installations; on stability requirements for floating installations, both intact and damaged, and on on-bottom stability. Reference is made to published papers, technical notes and state-of-the-art reviews.

4.1 LOADS

The principal loads to be taken into account, with notes on their characteristics, are defined below, grouped to facilitate reference in subsequent paragraphs and sections. Guidance is given to sources of data.

4.1.1 DEAD LOADS

These are fixed gravitational loads that can be varied significantly only by major alterations to the structure and/or the installed equipment. They comprise the weight in air of the entire structure including all secondary structures, the weight of all equipment and machinery installed on the installation and loads imposed by conductors, risers and any other normally fixed items which are supported by and transfer loads to the installation.

4.1.2 IMPOSED OR VARIABLE LOADS

Excluding only dead loads, hydrostatic loads and environmental loads, this group includes all other loads that the installation should be designed to withstand. Imposed loads are inherently variable in magnitude and/or disposition; in certain cases imposed loads have major horizontal components and in others allowance must be made for heavy impact loading. They include:

(a) Loads arising from the functional activities of the installation, e.g. drilling operations. These loads should be

assessed in consultation with the owner and any limits or precautions to be observed should be recorded in the Operations Manual.

(b) The weight of all variable and consumable stores and portable equipment used during operations; also crew members, furniture, effects and living stores. To be assessed as in (a) above.

(c) Berthing and landing loads. These include the loads imposed by vessels approaching, mooring and lying alongside, as detailed in Section 5.5, and helicopters landing and taking off, as in Section 5.3.5. Also loads arising from the transfer of persons and stores by the use of cranes or other means.

(d) Loads arising from moving a mobile installation and from maintaining it on station. These should include loads transferred to bollards, or equivalent, when the installation is under tow, being manoeuvred or moored; the thrusts developed by propelling machinery or thrusters and the hydrodynamic resistance due to movement through the water. Limiting weather conditions, sea states, speed, trim, load, etc., should be stated in the Operations Manual.

(e) Loads developed in a mobile installation from changes of trim and/or mode of support, including impact and straining forces caused by ballasting and changes to or from the floating mode. These loads should be assessed for the least advantageous conditions envisaged in the design of the installation.

(f) Thermal stresses and deformations due to different temperatures existing in different areas of the structure. Under Newfoundland and Labrador conditions large temperature gradients could exist due to the difference in temperature between external structure and adjacent internal structure in way of accommodation or machinery space at $30^{o}C$, and might occur simultaneously with severe wave conditions in a winter storm. A temperature gradient of approximately $55^{o}C$ may exist if hot oil is stored in a compartment that is externally in contact with sea water (references 1 and 26). Temperature effects on steel structures are referred to in references 23 and 24.

(g) Construction and installation loads imposed on fixed structures, including the loads imposed at various construction stages, lifting forces, load-out forces and forces experienced during launching, towing, up-ending and/or sinking: also loads exerted on the permanent structure by construction plant during construction, alteration and repair.

Construction loads should also include those applied by temporary auxiliary buoyancy chambers and similar equipment associated with a fixed installation and intended to facilitate movement to and installation at the permanent location.

The effects of immersion of a fixed installation to a greater depth than that at permanent location should be allowed for when necessary for deck erection. The factor of safety in these circumstances may be reduced, in consultation with the certifying authority, provided no damage is caused to the structure.

Limiting weather conditions, speeds and any other conditions that should not be exceeded during the movement and location of a fixed installation, should be determined and incorporated in the documents listed in Regulation 5 (1) (d) of the Offshore Installations (Design, Construction and Survey) Regulations 1981.

4.1.3 HYDROSTATIC LOADS

Hydrostatic loads act in a direction normal to the contact surface; they may be external or internal. Hydrostatic loads have been treated as a separate group because they can usually be determined with a high degree of accuracy, and when limit state design methods are used hydrostatic loads may justify a lower partial safety factor than items in the imposed load group.

4.1.4 ENVIRONMENTAL LOADS

4.1.4.1 General

Environmental loads are loads due to wind, waves, current, ice, snow, earthquake and similar environmental actions.

Unlike the majority of the loads so far described, environmental loads act horizontally or have major horizontal components.

Environmental loads are random in character and extreme magnitudes can be predicted only on a probability basis. Part I Section 2 sets out the minimum criteria to be taken into account, when dealing with the phenomena, both singularly and in combination.

4.1.4.2 Wind Loads

The wind force developed on an installation, or on elements

thereof, should be calculated using the methods recommended in Canadian National Building Code or on the basis of reliable wind tunnel observations which should always be made in respect of structures of unusual shape and/or size.

Dynamic wind loading effects due to vortex shedding, galloping and turbulent buffeting should also be investigated.

4.1.4.3 Hydrodynamic Loads

4.1.4.3.1 General

Hydrodynamic loads are loads induced by wave, current and relative motions between the structure and the surrounding water. When determining the hydrodynamic loads, the following parameters are to be taken into consideration:

- wave and current characteristics
- water depth
- size and shape of structure
- surface roughness of structure
- dynamic proportion of structure
- fluid-structure interaction
- hydrodynamic interaction between closely spaced structural members.

4.1.4.3.2 Wave Loads

The most significant contribution to the hydrodynamic loading on offshore structures is caused by the waves. The loads due to wave action may be split into two components, one first order oscillatory force which varies with the frequency of the waves, and one second order force. The latter component, often called the wave drift force, may be treated as a constant force in regular waves and as a slowly varying force in irregular waves. The absolute magnitude of the second order force is normally small compared to the first order wave forces, however, due to the slowly varying nature of this force in irregular waves, it may cause significant response of anchored structures due to the possibilities of exciting resonant frequencies of the system.

Wave force calculations should be based on wave theory appropriate to the depth of water, to the parameters of the wave and to the geometry of the structure itself. Waves also exert varying vertical forces and in certian circumstances, vertical and horizontal impact forces.

Guidance on wave theories and computation of wave loads may be obtained from recent publications (references 2-7, 22, 28).

References 2, 5 and 6 contain an extensive bibliography and an assessment of published drag and inertia coefficients for use in Morison's Equation and the conditions under which they are applicable. References 3, 4 and 7 deal with methods of computing wave loads on large bodies which operate in the diffraction regime of loading. It may be noted that such methods may also be used for calculating inertia coefficients for use in the Morison Equation, thus avoiding the necessity of using empirical formations for unusual shapes.

The methods developed in references 2-4 have been extended and validated experimentally in application to tethered buoyant platforms and other moored bodies.

4.1.4.3.3 Current Loads

Current loads should be computed in conjunction with wave loads by combining the water particle velocities due to wave and current. Suitable techniques are described in reference 12. Refer also to Sections 2.1.2.4.1 and 2.1.2.4.2 for the effects of currents on wave heights and increased 'drag'.

4.1.4.3.4 Wave Slamming Loads

Structural members in the zone between the trough and crest of the design wave may be subject to impact loads, usually termed 'slamming' (reference 8).

4.1.4.3.5 Vortex Shedding and Other Flow Induced Cyclic Loads

Cyclic loads due to vortex shedding, galloping and buffeting may arise from action of waves and current, alone or in combination. Assessment of the probability and magnitude of these loads is dealt with in reference 12. The effect of increase in hydrodynamic coefficients due to vortex shedding induced vibrations should be investigated.

4.1.4.3.6 Loads Due to Breaking Waves

For vertical and near vertical elements in the wave zone the effect of local shock pressure due to breaking waves should be considered.

The impact velocity for the shock pressure calculations

should be taken as that corresponding to the propagation velocity of the most probable largest breaking wave height for the actual condition.

4.1.4.3.7 Effects of Marine Growth

Marine growth increases the effect size and weight of the member colonized and may alter the values of drag and inertia coefficients (refernce 11).

4.1.4.3.8 Hydrodynamic Interaction Between Closely Spaced Elements

Where numerous elements are closely grouped, as f.inst. in a conductor array,jthe interaction effects due to shielding and solidification should be considered. The overall forces on the total group may be reduced, while the forces on the individual elements may be significantly increased relative tok the undisturbed flow condition. Similar effects will be present for f.inst. a riser attached to the platform leg. More information is given in references 9, 10 and 29-32.

4.1.4.3.9 Hydrodynamic Model Tests

As an alternative to, or to verify or supplement calculations, hydrodynamic forces may be derived from reliable test data and this should invariably be done when the presence of the installation is likely to alter significantly the profile of the wave(s).

4.1.4.4 Loads Due to Snow and Ice Accretation

Allowance should be made in accordance with the guidance given in Part II Section 2.1.2.8 of these notes.

For calculation of wind loads in the iced condition, the effect of increased windexposed dimensions and increased surface roughness due to the ice should be considered. In addition, the increase in gravity loads due to the ice, especially in a dynamic wind load analysis of wind sensitive structures, are of major importance.

4.1.4.5 Loads Due to Sea Ice

The ice loads on a structure depend on a number of factors:

- the geometry of the ice feature

- the mechanical properties of the ice feature
- the failure mode induced in a feature as a function of the structure geometry
- the degree of continuity between the ice feature and the structure
- the velocity of the ice feature
- the environmental forces available to drive ice feature
- the inertial effects in both ice and structure
- the change in water level when ice is frozen to the structure
- the change in temperature
- the mass of the ice feature

For further information on types of sea ice, see Part II Section 2.1.2.7.

4.1.4.6 Icebergs

The forces associated with iceberg-structure interaction depend on the following characteristics of the iceberg and the structure.

(i) Kinetic energy. The kinetic energy is due to the velocity of the iceberg and the masses of the iceberg and the water in motion with the iceberg. The force on the structure steadily increases to a maximum as a result of kinetic energy dissipation with the reduction in velocity of the iceberg.

(ii) Drag. The current and wind produces drag forces on the iceberg and these are transmitted to the structure after impact. The body of water in motion with the iceberg produces a varying drag after impact.

(iii) Compliance. The magnitude of the force depends upon the rate of change of velocity of the iceberg and would depend upon the movement of the iceberg after impact, due to crushing of the ice at the impact interface, local yielding (failure), and movement of the structure. The crushing of the ice depends on the shapes of the ice and the structure at the impact interface. The forces increase as a larger area of contact is established by local crushing and a more confined (multi-axial) state of stress is developed.

(iv) Stability. The iceberg could also rotate after impact and this could produce multiple impacts. The degree of rotation depends on the angular velocity of the iceberg before impact, the characteristics of the iceberg-structure interface and the relative position of the iceberg. Due to the irregular shape of the iceberg, it could increase in draft as it rolls and this must be considered while designing berms, glory holes, etc. as iceberg protection schemes.

(v) Dynamics. The transient nature of the force exerted by the iceberg must be considered in order to determine the dynamic response of the structure. The duration of loading could be comparable to the fundamental period of vibration of the structure and the inertial forces would be considable.

4.1.4.7 Earthquake Loads

4.1.4.7.1 General

The effects of earthquakes are to be considered for structures to be located in seismically active areas, see 2.1.2.9. In special cases earthquake effects may need to be considered in construction phase.

Two levels of earthquake activity are normally to be considered, i.e. the design level and the exceptional level.

a) the design level, corresponding to the design earthquake which is defined as the most probable severest earthquake expected to occur at the site with a recurrance period of not less than 100 years.

b) the exceptional level, corresponding to the maximum credible earthquake which could be expected to occur at the site.

The intent of the exceptional level is to ensure that the structure has sufficient energy absorbing capacity against reaching a progressive collapse limit state during rare intense earthquake motions. Normally, the maximum credible earthquake is not taken to be less than twice the ground motion of the design earthquake.

4.1.4.7.2 Ground Motion

The ground motions used as basis for design shall adequately represent the expected conditions at the site, both in terms of frequency content and energy distribution. See 2.1.2.9.

The effects of local soil conditions in amplifying or attenuating the ground motion and in altering the frequency content are to be studied in order to determine appropriate horizontal and vertical characteristic values of the ground motion.

The ground motion may be described either in terms of time histories or in terms of response spectra.

Standard spectra generally recognized as being valid for the region and the site conditions considered, may be considered when describing the ground motion.

The ground motion is normally to consist of three components which are to be applied simultaneously, i.e. in the two horizontal directions and in the vertical direction.

When the response spectra method is used the minimum values of ground motion in the three directions are;

- 100 percent in the horizontal direction (i.e. the principal axis) considered most unfavourable to the structure.
- 70 percent in the orthogonal horizontal direction,
- 50 percent in the vertical direction.

Where deemed necessary, appropriate higher percentages are to be used.

The ground motion components to be used in time history analysis are to be derived using recognized procedures.

The statistics and procedures used, and the investigations made for establishing the characteristics of the ground motion to be used as basis for design are subject to acceptance by the certifying authority.

4.1.4.7.3 Other Effects:

The effects of tsunamis are to be considered for structures located in shallow water.

The effects of earthquake generated acoustic shock waves are to be considered where such effects will result in additional loads, e.g. sub-sea compartments that are not fully flooded.

4.1.4.7.4 Structural Analytical Model:

The analytical models used for determination of earthquake response should represent the structure- soil- water system in a realistic manner as regards to distribution of stiffness, mass and damping.

Assumptions made as regards nonlinear behaviour are to be carefully examined with respect to material properties and structural detailing.

Normally, the dynamic characteristic of the structure and its foundation should be determined using a three dimensional analytical model. A two dimensional model may be used provided the torsional response and the response of cantilevered structures, e.g. decks on gravity structures, are properly accounted for.

Parametric studies are to be carried out where deemed necessary for proper determination of loading effects. The use of partitioned damping is recommended for parts of the structure-soil-water system having relatively large damping compared to the damping in the structure, e.g. the material and radiation damping in the soil.

4.1.4.7.5 Methods of Analysis:

The response analysis may be accomplished using any recognized method applicable for offshore structures, e.g. response spectrum methods or time history methods.

When using the response spectrum method as many modes should be considered as required to provide at least 90 percent of the total energy of all modes. A minimum of six modes with the highest energy content should be considered. Unless otherwise agreed, the maximum total response S is not to be taken less than

$$S = s_n + \sqrt{\sum s_i^2}$$

where s_n is the modal response yielding the largest contribution and s_i the individual modal response of the other modes.

When the response is carried into the inelastic range, time history methods are normally to be used.

Static equivalent methods are not to be used as the basis for design of structures for which sufficiently accurate dynamic analyses can be carried out.

For structures for which a meaningful dynamic analysis cannot be carried out for the maximum credible earthquake, acceptable safety against progressive collapse may normally be assumed if it can be verified that the structure remains stable when subjected to a quasistatic loading that yields deformations approximately twice those of the design earthquake.

4.1.4.7.6 Equipment Response:

The purpose of equipment response analysis is to determine the loads to be used in the design of the equipment support structures and the fastening arrangements, and in the checking of the safety of equipment.

The dynamic response of equipment and equipment supports is to be analyzed using recognized procedures and methods. The vertical and horizontal support accelerations should be combined in a rational manner so as to arrive at realistic loading effects.

The criteria for equipment survivability are subject to agreement with the certifying authority.

4.1.5 DYNAMIC LOADS

Dynamic loads, due to the momentum developed in a structure, or part thereof, when responding to imposed forces, should be taken into account where likely to be significant.

Dynamic response, including methods of assessing its likely significance in various circumstances, is dealt with in reference 12.

The dynamic response of tethered buoyant platforms is dealt with in reference 10.

4.1.6 LOADS IN COMBINATION - EXTREME CONDITIONS

4.1.6.1 Fixed Installations

Under the most severe combination of environmental conditions set out in Section 2, an installation should have adequate resistance to sliding, overturning and structural failure due to over-stressing, taking account, where appropriate, of the augmented stresses due to dynamic loading.

Each variable load should be assumed to have that value, and position, within its possible range that will minimize the margin of resistance to the mode of failure under examination. Advantage may be taken of any possibility of reducing loads significantly by suspending operations or by taking any other measures within the control of the Installation Manager, provided clear instructions to this end are set out in the Operations Manual.

For each possible mode, the structure, including the underlying sea-bed, should be capable of developing the necessary resistance without exceeding the safety margins set out in this document.

4.1.6.2 Mobile Installations

A mobile installation temporarily supported by the sea-bed should be considered as a fixed installation and should be capable of complying with the requirements of 4.1.6.1 unless the design is such that it can, and will, change safely, to the floating mode within the shortest normal warning period of severe weather.

A mobile installation afloat should be capable of withstanding the environmental conditions calculated to develop the most severe stresses in the structure unless such conditions are likely to occur less frequently than once, on average, in any period of 100 years.

Under the above conditions, each variable load should be assumed to have that value, within its possible range, and such position where this can be varied, as will develop the most severe stresses in the structure. But advantage may be taken of any capability to alter loads, trim, freeboard, orientation, mooring systems and other protective measures that can minimize loads on the installation structure provided instructions to this end have been included in the Operations Manual.

Under these conditions the installation should be capable of developing adequate resistance to structural failure without exceeding the limiting stresses or other safety margins set out in this document.

4.1.7 LOADS IN COMBINATION - OPERATING CONDITIONS

The owner may specify the most severe environmental conditions during which operations will be allowed to continue. Otherwise as 4.1.6.1 and 4.1.6.2 above.

In some cases, e.g. large production platforms where drilling has been completed, the 'Operating' condition may not be significant.

For a mobile installation in transit the Operations Manual should state the maximum load that may be carried and its disposition on board and the draft appropriate to specific wind/sea states. Where mobile platforms are intended to be in production under extreme weather conditions these should be considered the 'Operating' conditions.

4.2 DESIGN CRITERIA

This section contains information on recent investigations and validated research results that should be applied, where appropriate, in the fields of dynamic response, buckling, fatigue and the design of grouted pile/jacket connections.

Fixed steel installations are covered under main heading 4.2.1, mobile steel installations under 4.2.2, and concrete installations in 4.2.3.

4.2.1 FIXED STEEL INSTALLATIONS

Design is to be based upon methods as set out in Canadian Standards Association design standards for steel structures (CAN3-S16.1-M78 and CAN3-S6-M78). Other design concepts may be used if it can be demonstrated that the margin of safety achieved will be no less than that attributed to established traditional methods.

4.2.1.1 Structural Analysis

Established structural analysis techniques are well documented in standard textbooks, supplemented by numerous authoritative technical papers on specialized aspects of design. It would neither be practicable nor useful to attempt to list all relevant references. Any recognised design method may be used provided it is appropriate to the problem under examination and that it can take into account the most recently available information on structural behaviour and material properties.

4.2.1.2 Dynamic Response

Load augmentation due to dynamic response to variable forces becomes of increasing importance as installations are designed to be located in deeper and deeper waters and their natural

period becomes nearer to that of waves transmitting significant amounts of energy. Where necessary, in addition to static and quasi-static calculations, a dynamic analysis should be undertaken. Guidance on the calculation of dynamic response, including methods of assessing its probable significance, is contained in reference 12.

4.2.1.3 Design Requirements

The structure shall be designed to be serviceable during the design period and safe from collapse during construction and during the design period as specified in Canadian Standards Association publications CAN3-S16.1-M78 Steel Structures for Buildings - Limit States Design and CAN3-S6-M78 Design of Highway Bridges.

Where the structural element, or type of loading, is not covered by the CSA publications, a rational analysis should be used to achieve appropriate load combinations corresponding to a safety level not less than in the standards quoted.

4.2.1.4 Joint Design for Welded Tubular Steel Structures

(a) Design strength formulae

Design strength formulae for simple tubular joints are given in reference 23.

(b) Geometry

Where braces are separated, i.e. there is no overlap, the minimum distance between the toes of the attachment welds should be not less than 50 mm. When braces are overlapped the overlap should preferably be proportioned for the transfer of 50 percent of the total load. If overlap is large or complete, the overlapped brace is effectively a chord member and must be considered as such in the analysis of the joint.

(c) Reduction of Peak Stresses

Internal gusset plates or external ring or longitudinal stiffeners can be used to reduce the peak stresses at tubular joints and to increase static strength. For stiffeners to be effective, the location is critical. For each specific configuration it is advisable to determine by model tests or a photo-elastic investigation, or a finite element analysis, the influence of the stiffeners on the stress distribution. It should be noted that longitudinal stiffeners extending along the bore of a brace from the joint to the chord member can have a negligible effect on the peak stress in the chord member, and that fillet welds at the outer ends of such stiffeners have been the sites of fatigue failures in service.

4.2.1.5 Buckling

Components of offshore steel platforms and mobile drilling rigs which may be susceptible to buckling failure include the following:

(i) Slender chord and bracing elements of the structural framework, usually of round tubular construction; consideratin must be given to behaviour under axial compression possibly combined with bending, and external pressure.

(ii) Flat stiffened panels, as occur in platform decks and supporting deep girders: buckling failure may occur under compressive, bending and shear loads.

(iii) Ring-stiffened cylindrical shells: buckling may occur under external hydrostatic pressure, axial compression, bending, shear or some combination of these loads.

(iv) End-closures to cylindrical shells, usually of hemispherical, torispherical or conical form: buckling may occur under external pressure.

(v) Large diameter orthogonally stiffened cylinders: consideration must be given to buckling under axial compression or bending loads which may be combined with external pressure and possibly shear.

Buckling failure will generally take place inelastically and may be strongly influenced by manufacturing imperfections including particularly initial deformations and residual stress caused by misalignment, welding and cold-forming. Most offshore platforms are highly redundant structures whose ultimate strength will depend not only on the buckling strength of individual elements but also on pre-collapse loss of stiffness and post-collapse load-carrying capacity of such elements.

4.2.1.5.1 Tubular Members Under Axial and Bending Loads

Guidance on design of axially compressed unstiffened tubes of low diameter/thickness ratio ($D/t < 70$) is given in references 23, 33 and 34 or references 24 and 25. Guidance on design of tubes under combined axial and bending loads using interaction formulae is given in references 23 and 33. Guidance for the design using first yield criterion is given in references 24 and 36.

Consideration must be given to local buckling of tube walls if D/t exceeds a certain level, (given by reference 23 as 60, by reference 33 as 0.11 $E/6_6$, and by reference 24 as 0.088 $E/6_y$), where E = Young's Modulus and 6_y = yield stress. For thinner walled tubes in which D/t exceeds these limits, the local buckling stress σ_c for a thin-walled cylinder under axial compression may be substituted for the yield stress σ_y in the column design formulae: empirical formulae for σ_c are given in references 23, 34 and 33. Because of the catastrophic post-collapse loss of load-carrying capacity in thin-walled, high-strength tubes ($D/t > 0.2\ E/\sigma$) and their sensitivity to demage in the form of small dents (reference 37), particular care should be taken when considering the use of such members in the primary structure.

Information on the effects of lack of straightness and residual stresses in tubular members may be found in references 37 and 38. Some guidance on pre-buckling loss of stiffness, post-collapse load-carrying capacity and dmanage effects in tubular beam-columns is contained in reference 37. Design guidance on beam-columns of rectangular box-section and open cross-section is provided in references 33, 24 and 39.

4.2.1.5.2 Flat Stiffened Panels and Plate Girders

Failure of a flat stiffened panel under uniaxial or biaxial compressive loads may involve:

(i) Local buckling of rectangular plate panels between stiffeners:

(ii) Interframe flexural buckling of stringers (normally aligned in the direction of dominant compressive load) and attached plating between heavier, more widely spaced transverse stiffers;

(iii) Lateral-torsional buckling of stiffeners;

(iv) Overall instability involving bending of longitudinal and transverse stiffeners.

Failure modes (ii), (iii) and (iv) may be strongly affected by loss of plating stiffness caused by buckling and yielding of plate panels: guidance on effective plate stiffness and strength under uniaxial and biaxial compression may be found in references 24, 40 and 41. An understanding of failure mode (ii) (interframe flexural buckling) may be obtained from tests and parametric studies described in references 17 and 42 to 45. Torsional buckling of stiffeners is normally avoided by conservative proportioning of stiffener sections: guidance is contained in references 24, 33, 39 and 46. Overall buckling

may be suppressed by providing sufficient transverse flexural stiffness to ensure that interframe buckling precedes overall instability. Guidance on detailed design procedures for stiffened panels under compressive load are contained in references 24 and 46.

Design of stiffened and unstiffened plate girders under shear load, possibly combined with in-plane compression or bending, may be based conservatively on elastic buckling criteria by ensuring, with appropriate margins of safety, that:

(i) Combined stresses do not cause yield;

(ii) Stiffeners have sufficient flexural rididity to supress overall buckling;

(iii) The elastic buckling stress of plate panels between stiffeners is not exceeded.

Design procedures based on this approach are described in references 24 and 39. Design may be based, alternatively and less conservatively, on plastic collapse criteria allowing for the post-buckling strength of slender girder webs associated with diagonal tension: guidance on this option is included in references 39 and 46. Care must be taken to avoid local web buckling under high concentrated loads: guidance is included in references 39, 40 and 47. A recent general review of girder buckling behaviour and design methods is contained in reference 41. Recent data defining the stiffness, strength and post-collapse behaviour of plates under shear and combined loads may be found in reference 49.

4.2.1.5.3 Rig-Stiffened Cylinders

Failure of a ring-stiffened cylinder under external hydrostatic pressure may occur in any of the following modes:

(i) Interframe shell buckling which may be either axisymmetric or "lobular" in form, i.e., may involve either a single lobe or several buckling waves round the circumference of the shell between frames: design is normally based on empirical lower-bound curves as outlines in references 23, 24 and 25.

(ii) Lateral-torsional instability of ring frames, normally avoided by conservative proportioning of frame cross-sections (references 24, 25, 33 and 39).

(iii) General instability involving bending of ring frames: design is normally carried out by proportioning frames so that general instability occurs as a substantially higher pressure than interframe buckling (references 23, 24, 25 and 33).

The latter form of failure is particularly sensitive to departures from circularity in ring frames and to residual stresses caused by cold-forming of frames: guidance on the effect of imperfections, both within and outside normal tolerances, may be found in references 17, 50 and 51.

For design purposes interframe buckling of a ring-stiffened cylinder under axial or bending load may be treated conservatively as that of an unstiffened cylinder of lingth equal to the spacing of frames or diaphragms. This form of failur is very sensitive to short-wavelength imperfections (reference 52): imperfection sensitivity is, however, reduced by the presence of closely spaced ring frames (reference 53). Sizing of ring frames to avoid overall instability may be based on elastic buckling analysis (reference 24) or test data (reference 53). In the case of long cylinders, consideration should be given to possible interaction between local shell buckling and overall column instability (reference 24).

Guidance on desion of ring-stiffened cylinders under axial or bending load combined with external pressure may be based on interaction formulae in references 23, 24 and 53).

4.2.1.5.4 Hemispherical, Torispherical and Conical End-Closures

Guidance on design mehtods against buckling under external pressure is found in references 25 and 54. The basis of these methods is explained in reference 55. The sensitivity of spherical shells to imperfections in the form of local flattening should be noted (Reference 56).

4.2.1.5.5 Orthogonally Stiffened Cylinders

Buckling of orthogonally stiffened cylinders under axial compression or bending load is analogous with that of orthogonally stiffened flat panel under uniaxial compression: possible failure modes correspond to those described in Section 4.2.5.1.2.

Overall buckling should normally be excluded by spacing and proportioning ring frames or diaphragms so that, on the basis of elastic buckling analyis interframe buckling modes precede overall modes by a substantial margin: guidance on formulae

for the evaluation of elastic buckling, allowing stiffener eccentricity, may be found in references 17 and 24.

Torsional buckling of stringers should be avoided by careful proportioning of stiffener sections (See Section 4.2.1.5.2 and references 23, 24, 25 and 39).

For design purposes buckling of a stringer-stiffened shell between transverse frames or diaphragms may be treated conservatively as that of a flat-stiffened panel (See Section 4.2.1.5.2): this approach (reference 24) is reasonable for short cylinders (closely spaced rings or diaphragms) but becomes very conservative for long cylinders (reference 58). Similarly the local buckling strength and stiffness of curved plate panels between stringers may be examined conservatively as that of equivalent flat panels: more accurate estimates of the strength, stiffness and imperfection sensitivity of axially compressed curved panels in the elasto-plastic range may be made using data contained in reference 59. In the case of long cylinders consideration should be given to possible interaction between local shell buckling and overall column instability (reference 24).

The buckling strength of an orthogonally stiffened cylinder under external hydrostatic pressure may be estimated conservatively, ignoring the influence of stringers, as that of a ring-stiffened cylinder (See Section 4.2.1.5.3). For the case of external pressure combined with axial compression or bending, interaction formulae may be used as for a ring-stiffened cylinder.

4.2.1.6 Stiffness

The maximum permissible deflections will depend on the nature of the structure and the service for which it is intended. Consideration should be given to the possibility of elastic instability of the structure as a whole; deflections of individual members should be limited so as to avoid such instability.

4.2.1.7 Vibrations

Precautions should be taken in the design against the possibility of excessive structural vibration being induced by machinery. This entails investigation of the natural frequencies of the members and of the sources of excitation.

When it is not possible to be certain that excessive vibrations will not occur a survey should be made of the

finished installation during initial service and any local or general resonance which could be damaging should be identified and if necessary corrective action taken. (See also 5.9.)

4.2.1.8 Stress Concentrations

The design and fabrication or construction of all structural details should be such as to minimize stress concentrations.

4.2.1.9 Progressive Collapse

The installation should be so designed that accidental damage to part of the installation will not lead to progressive collapse of the whole installation.

The progressive collapse limit state is to be considered for possible accidental (exceptional) loads, against which sufficient local strength cannot be provided by reasonable means, or against which increased local strength would reduce the safety against overall collapse of the structure.

The extent of the damage may either be defined, or determined on the basis of a defined "accident". When relevant, the effects of the accidental load itself on the intact part of the structure are to be considered.

For steel structures load and material coefficients may be taken equal to unity, i.e. all design values may be taken equal to the characteristic values.

For concrete structures, the possibilities and effects of rupture of critical sections are to be examined carefully. If the rupture of a section may cause a catastraphic collapse, the section or member, should, if possible be designed so that a possible failure will develop in a ductile manner. For concrete structures, material coefficients may be taken as 1.1 for concrete and 1.0 for reinforcement.

4.2.1.10 Fatigue - General Considerations

The following procedures on fatigue apply to all types of welded offshore steel structures.

(a) Background

An offshore installation is exposed throughout its life to environmental loadings which causes cyclic stress variations in its structural members resulting in the

possibility of fatigue cracking in these members. Wave loading e.g. over a 20 year service life (which means here the anticipated operating life) will result in about 10^8 cycles of stress variation and it is this wave action which is the source of potential fatigue cracking. However, any other source of cyclic loading could also contribute to fatigue damage and should therefore be considered. Unless otherwise agreed with the certifying authority, the calculated fatigue life derived by the methods below should be not less than 20 years, or the required service life if this exceeds 20 years.

(b) Scope of fatigue analysis

To ensure that the structural members of the installation fulfill their intended function, a fatigue analysis should be carried out for each individual structural member which is subjected to fatigue loading. In this context it should be noted that, in any element or member of the structure, every welded joint or other form of stress concentration is potentially a source of fatigue cracking and each should be individually considered.

(c) Fatigue loading

In assessing fatigue performance all types of cyclic loading must be considered. It should be noted that cyclic loading from different sources may be significant at different phases of the life of a structure (e.g. construction, transport, installation, in-service).

Some important sources of cyclic loading are:- Waves (including those which cause 'slam' and variable-buoyancy effects), wind, especially when vortex shedding is induced e.g. on slender members), currents (where these, by reason of non-linearity, influence the forces generated by waves), ice (breakup as ice moves around the structure) and mechanical vibration (e.g. caused by operation of machinery). In hydrodynamic loading, the effects of shielding and interference of members in close proximity need to be taken into account.

It should also be noted that static loading may be significant in the fatigue calculation if benefit from stress relief is contemplated.

(d) Basis of fatigue analysis

In order to evaluate fatigue lives, it is necessary to establish the long term distributions of stress range taking into consideration all stress variations occurring during the life of the structure which have magnitudes and numbers large enough to cause fatigue effects.

To calculate the long term stress range distributions, either a deterministic or spectral analysis procedure may be used, whichever is considered the more appropriate for the particular circumstances, as agreed with the certifying authority.

Due consideration should be given to the orientation of the structure in relation to the distribution of directions of the environmental loading conditions causing cyclic loading; it should be assumed in each that wind, waves, current and ice all act in the same direction.

(e) Fatigue life

The design fatigue life is assessed from the calculated long term stress range distributions.

The resultant calculated fatigue lives enable critical parts to be ranked in terms of fatigue sensitivity thus establishing a priority basis for developing a selective inspection programme to be followed during the service life of the installation.

Operating experience and research developments during the service life may indicate a need to update the fatigue life calculations and in this event discussions should be held with the certifying authority.

(f) Factors on fatigue life

The design is based on S-N Curves. For a critical member, e.g. one whose sole failure would be catastrophic, an additional factor on life should be considered, in agreement with the certifying authority. In defining this factor on life, account should be taken of the accessibility of the joint and the proposed degree of inspection as well as the consequences of failure.

Because the calculated life is sensitive with the estimates of stress ('power of three'), particular care must be taken that stresses are not underestimated. Should the certifying authority consider that uncertainties exist in these and other relevant matters, it may impose additional factors.

(g) Factors influencing fatigue behaviour

For welded steel structures it has been established that the fatigue life is normally governed by the fatigue behaviour of the joints, including both main and attachment welds. The best fatigue behaviour will be obtained by

ensuring that the structure is so detailed and constructed that stress concentrations are kept to a minimum and that the elements may deform in their intended ways without introducing secondary deformations and stresses due to local restraints. Stresses may also be reduced by increasing the thickness of parent metal or weld metal; this should improve fatigue life but it should be remembered that fatigue strength tends to decrease with increasing thickness.

4.2.1.11 Stresses to be Considered

The procedure for the fatigue analysis is based on the assumption that for as-welded structures it is only necessary to consider ranges of cyclic principal stress in determining the endurance (i.e. mean stresses are neglected). However, in stress relieved joints subjected to stress cycles which can be clearly demonstrated to be partly compressive in nature it may be assumed that only 60 percent of the compressive component is damaging; thus the relevant stress range becomes equal to the tensile component plus 60 percent of the compressive component. In either case, therefore, residual stresses need not be included in the stress calculations.

In situations where the potential fatigue crack is located in parent material (e.g. at a weld toe) the relevant stress is the range of principal stress adjacent to the potential crack location, stress concentrations being taken into account as indicated below. For cracks which might occur in a weld throat the relevant stress range is the vector difference of the greatest and least vector sums of the shear stresses in the weld metal, based on the effective throat dimensions and assuming that none of the load is carried in bearing between parent materials.

4.2.1.12 Grouted Pile/Sleeve Connections

The strength and stability of a jacket structure depends on the integrity of the connection between the supporting piles and the jacket legs or pile sleeves. In template and in skirt piled structures, some of the piles may pass through the legs of the structure and the primary connection is then normally formed by welding the pile to the structure above water. In other cases, particularly for structures in deeper water, a number of sleeves are provided at the lower end of each leg and grouted connections are used to transmit all loads between the structure and its piles. The following design procedures are intended to cover all instances where a grouted connection forms the primary means of load transfer between the structure and the piles.

The following recommendations are derived from model and full scale laboratory tests on specimens representing the geometry of typical connections used in other areas. The design procedure recommended is similar to that presented in reference 13. Further background information can be found in references 14 and 15.

4.2.1.12.1 Factors Affecting the Strength of a Grouted Connection

The following principal factors have been shown to affect the strength of a grouted connection:-

(a) Grout compressive strength and elastic modulus

(b) Tubular and grout annulus geometries

(c) Outstand and spacing of mechanical shear connectors

(d) Grouted length to pile diameter ratio

(e) Surface connection of tubulars

(f) Long term grout shrinkage or expansion.

Items (e) and (f) are significant only for plain pipe connections.

4.2.1.12.2 Use of Mechanical Shear Connections

It is strongly recommended that mechanical shear connectors are used, since their presence can result in connection strengths of several times that of plain pipe connections, increases in the reliability of the connection, and elimination of any effect of long-term grout shrinkage. Mechanical shear connectors may take the form of continuously welded bars or weld beads attached to the steel surfaces in contact with the grout and may be arranged in hoop or helical formation. Other forms of shear connection require special consideration.

Where the design of the grouted connections take account of the presence of shear connectors, the following requirements should be noted-

i) Shear connectors should be present on both the pile and sleeve surfaces which are in contact with the grout

ii) The shear connector spacing should be uniform along the length of the connection

iii) The outstand and spacing of shear connectors on the sleeve and pile should be the same

iv) For driven piles, shear connectors on the pile should be applied to sufficient length to ensure that, after driving, the length of pile in contact with the grout has shear connectors

v) Shear connector cross-section and welds on each grout/steel interface should be designed to transmit the total load applied to the grouted connection.

4.2.1.12.3 Applied Stresses

In calculation of applied bond stresses due account should be taken of the distribution of overall structure loads between various piles in a group or cluster. The design loads for the connection shall be the highest calculated load with due consideration of the range of possible in-situ soil stiffness.

Bond stress is defined as the applied axial load divided by the pile to grout nominal contact area.

4.2.1.13 Grout Materials and Grouting Procedures for Grouted Pile/Sleeves Connections

4.2.1.13.1 Materials

The types of materials which may be used for grouted connections include-

i) Portland cement grouts with or without inert fillers mixed preferably with fresh water, although seawater may be used. There may be special circumstances where the use of seawater is undesirable because of corrosion and other durability effects.

ii) Fresh water/high alumina cement grouts providing that, to take account of the conversion process, the design is based on the minimum strength appropriate to the curing temperature, service temperature and water: cement ratio. In this respect the water:cement ratio should not be greater than 0.4.

Admixtures may be used to improve properties of the slurry or set grout provided that it is satisfactorily established that they have no harmful effect on the performance of the connection. The use of calcium chloride or admixtures containing significant levels of chloride ion are not permitted. Guidance on the strength-time characteristics of typical grout mixes can be obtained from reference 16.

4.2.2 MOBILE STEEL INSTALLATIONS

Subject to any requirements set out in this publication, any installation intended to operate or manoeuvre afloat should be designed in accordance with the latest rules for mobile installations published by any one of the appointed certifying authorities.

Note: It is not necessary that the design rules should be those of the authority selected by the owner to certify the installation.

Account should be taken of the guidance on fatigue analysis set out in 4.2.1.10 and 4.2.1.11.

4.2.2.1 Bottom Supported Units

Platform elevation and descent: Any elevating system should be capable of adequately lifting and supporting the installation when it is in use. It should be designed to pre-load the foundation for the design conditions; the jack or jacks acting on any legs should be capable of applying a load in a downward direction of at least the maximum load for which the leg has been designed. For each leg there should be provided an indicator which will register the load on it at any time. Particular care should be taken in materials with a view to avoiding failure by fatigue or brittle fracture. A suitable means of de-icing should be incorporated, if needed.

The damage records of mobile platforms indicate that foundation problems should be addressed carefully. The suitability of a certain rig for a given location is dependent on, among other things, its foundation design and factors such as loading conditions, soil conditions, depth of leg penetration, stability considerations, necessary preload, etc. (refer Section 3.1.5.1).

Elevating mechanism attachments: where clearance between the legs and their housings allows a degree of relative movement between legs and platform, the arrangement of the leg/platform attachment should be based either on designs already proved satisfactory in service or justified by

calculations that take full account of the effects, including inertia effects, of the lack of fixity at the attachment. The design should be such that modifications (e.g. the fitting of shims) are unnecessary to combat storm conditions.

Legs: Legs should be capable of accommodating static and dynamic forces and movements resulting from the following circumstances:

(i) Location moves: the legs should be able to withstand the bending moment induced by rolling or pitching to 10^{0} single amplitude at the natural period of the unit, plus a suitable wind moment and 120 percent of the gravity moment due to the angle of heel. The procedure to be used in the event of these conditions being exceeded should be recorded in the Operations Manual. Legs should also be able to withstand the dynamic loads that may be caused by their unsupported length moving through the water. Design figures should be supported by a model test or experience with similar units.

(ii) Lowering of legs to bottom: legs should be able to withstand the shock of touching bottom while the unit is afloat and subject to wave motions. The maximum design roll and pitch angles while lowering legs should be indicated in the Operations Manual; it should be stated that the legs must not be permitted to touch bottom when unit movements exceed the stated limitations.

(iii) Legs with hull in elevated position: the legs must be able to withstand the loads due to the most adverse conditions of loading, including any effects resulting from clearance between the legs and their housings or attachments.

4.2.3 CONCRETE DESIGN

For fixed offshore installations, intended to be afloat only whilst under construction and during transit from building site to final location, during the transit period and until permanently settled on the sea-bed, the installation should not be exposed to conditions more severe than those envisaged in the design for this period.

Experience gained in classifying ships and barges would be useful for certifying concrete installations intended to operate afloat. Good concreting practice will produce sound concrete irrespective of the type of structure into which it will be incorporated. Loads and stresses may be assessed by established ship design techniques, allowance being made for the behaviour characteristics of the material.

4.2.3.1 Codes of Practice

Subject to any over-riding requirements set out in this publication, the materials, construction, design and testing of concrete offshore installations should be in accordance with the recommendations in CSA publications CAN3-A23.1-M77 - Concrete Materials and Methods of Concrete Construction, CAN-3-A23.2-M77 - Methods of Test for Concrete, CAN3-A23.4-M78 - Precast Concrete - Materials and Construction, and CAN3-A23.3-M77 - Code for the Design of Concrete Structures for Buildings, supplemented by reference 27.

If owner and/or certifying authority are of the opinion that a specific design problem is not adequately covered by the above documents they may, by agreement with the Minister , apply either the recommendations contained in some other recognized document or resolve the problem by the application of first principles.

4.2.3.2 Design Considerations

(a) Concrete strengths

In all cases a 'designed mix' should be used. For reinforced and prestressed concrete the specified 28 day strengths should not be less than:

30 N/mm^2 for internal atmospheric zones;

40 N/mm^2 for concrete in the external atmospheric splash and submerged zones and for prestressed concrete;

50 N/mm^2 for concrete subject to severe abrasion.

The specified cylinder strength of unreinforced concrete should be appropriate to its use with particular attention paid to the durability requirements for concrete which will be exposed to aggressive physical or chemical conditions.

In cases where structural parts will be subjected to loads earlier than 28 days, the design strenth is to be based on the characteristic strength corresponding to the actual age.

(b) Welding reinforcement

Welding should be avoided as far as possible but where essential it should be carried out in accordance with recommended practice.

Welding reinforcement under controlled conditions should not reduce static strength but if subsequently subjected to

cyclic loading the fatigue life will be reduced due to the development of stress concentrations and 'hot spots': in the absence of authoritative data the fatigue life should be calculated on a conservative basis.

(c) Deflection

Long-term deflection calculations should assume that the maximum level of stress is applied constantly. This will allow for creep deflection and has been found to yield reasonably accurate answers. Where the connection between the structure and a well head necessitates special deflection or settlement requirements, these should be stated and considered in design. Sufficient data should be provided on the strength and stiffness of conductor tubes and walls to enable the interaction between these and the structure to be assessed accurately.

(d) Oil containment

In members with re-entrant angles, where localized stress concentrations are high, it may be impossible to limit the size of cracks. Therefore to prevent leakage, the oil storage system should be designed so that the pressure of oil within the structure is always below that of the surrounding sea. In the operating condition the pressure differential should be equivalent to at least one metre head of sea water.

The temperature of crude oil entering the storage system allowing for any cooling which may be provided for in the early stages of the process plant, should be stated in the design brief.

When oil storage is divided into compartments the effects of differential filling should be considered. Where uniform filling is required, the oil storage control system should be designed so that accidental differential filling cannot occur.

Where the wells pass through the structure, the transfer of heat from the wells through the surrounding water to the structure itself should be considered.

Oil control systems. Oil control systems should be designed so that any changes in temperature or pressure do not cause overstressing of the structure.

(e) Zones of exposure

External zones

i) Submerged zone: that part of the structure below the splash zone, including buried parts.

ii) Splash zone: that part of the structure ranging between 65 percent of the 100 year (average) wave height superimposed on the highest astronomical tide and 35 percent of this wave height below the lowest astronomical tide.

iii) Atmospheric zone: that part of the structure above the splash zone.

Note: Structures continuously exposed to salt-laden air are often as vulnerable to attack as those in the splash zone.

(f) Concrete cover to reinforcement and prestressing tendons-

The following figures should be used:

The nominal cover to all reinforcement and prestressing tendons should be not less than the greatest of the following:

(1) The cover shown in the table below:

Table 4.1

Zone	Nominal cover (mm) Reinforcement	Prestressing tendons
External:		
Submerged	60	75
Splash	75	100
Atmospheric	75	100
Internal:		
Submerged	60	75
Atmospheric	75	100

The cover quoted is for concrete having a characteristic strength not less than 40 N/mm^2.

(2) 1.5 X the nominal maximum size of aggregate.

(3) 1.5 X the maximum diameter of reinforcement or prestressing tendons.

(4) For bundled bars the greater of 1.5 X the diameter of the largest bar in the bundle or the diameter of the equivalent bar; but not more than 100mm. (The equivalent bar is a single bar having the same area as the bundle of bars).

4.2.3.3 Removal, Damage or Abandonment

Reference should be made to Section 5.13, which deals with measures that should be incorporated to facilitate removal when and as necessary.

4.2.4 Instrumentation

In the case of fixed installations very severe loads may be developed during movement of the installation from the construction site to its permanent location and during the operations associated with setting the installation on the sea-bed. To confirm the design assumptions suitable instruments should be installed to monitor the behaviour of the structure and its foundations during the initial construction and operating stages and subsequently for general monitoring and to assist in the assessment for re-certification.

The level of instrumentation would depend on the installation and location and should be approved by the certifying authority.

(a) In general there shall be provided on every installation available means for ascertaining at any time-

i) the wind speed and direction;

ii) the air temperature;

iii) the barometric pressure;

iv) the sea state; and

v) the ice conditions

(b) There shall be provided on every offshore installation suitable means for ascertaining at any time-

i) the strain, vibration and acceleration;

ii) the water, hydrodynamic and shock pressures;

iii) corrosion data; and

(c) There shall be provided on every fixed and compliant installation suitable means for ascertaining at any time-

i) the pore and earth pressures; and

ii) the settlement.

(d) There shall be at all times in force in respect of every offshore installation a scheme providing for the systematic ascertainment and recording of:-

i) the matters mentioned in paragraphs (a), (b) and (c) above, and

ii) the visibility and the cloud base and cover.

4.3 STABILITY (MOBILE UNITS)

4.3.1 FLOATING INTACT STABILITY

(a) Wind Velocities

Units should possess adequate stability to withstand the overturning effect produced by a steady wind for all conditions afloat, whether in transit, being placed on or removed from station, or when on station. For the operating loading condition the wind velocity to be used in the stability calculations should be the highest of the maximum permissible limiting wind velocities but not less than 35 meters per second (about 70 knots). For the extreme loading condition, a steady velocity of not less than 50 meters per second (about 100 knots) should be assumed. The maximum permissible wind velocities, design limits of roll, pitch and heave, and also instructions for carrying out ballasting or other changes in the mode of operation, should be included in the Operations Manual and any changes in the mode of operation of the unit (such as changing the draft, redistribution of loads, equipment or ballast) should be capable of being made within a maximum period of three hours. A wind velocity of 35 meters per second should be assumed for those temporary conditions.

(b) Centre of Gravity

The location of the centre of gravity of the unit may be determined by calculations or by an inclining experiment carried out on the unit, or on a similar unit but modified by calculated variations for the subject unit.

(c) Effect of Mooring System

Mooring systems which may adversely affect stability should be taken into account, but no allowance should be made for any advantages which may be claimed for such systems. Some relaxation may be allowed for units which are of the single buoy mooring type.

(d) Heeling and Righting Moments

Wind heeling moments for units with ship-shaped hulls may be assumed to vary as the cosine of the vessel's heeling angle. For all other units the moments should be calculated for a sufficient number of angles of heel to define the wind heeling moment curve. For all units the righting moments should be calculated for a sufficient number of angles of heel to define the righting moment curves. These curves should be produced for the full range of operating conditions and should show:

(i) For ship and barge-shaped units and bottom-supported units in the floating mode, the area under the righting moment curve to the second intercept or the angle of downflooding, whichever is less, should be not less than 40 percent in excess of the area under the wind heeling moment curve to the same limiting angle. In addition to the above requirements, the units should have positive stability, neglecting wind effects, for the following minimum ranges of heeling angles:

for units operating in the floating mode 0^0-35^0;

for field moves where transit voyages are expected to be completed in favourable weather within 12 hours or within the limits of accurate favourable weather forecasts 0^0-20^0;

for long moves 0^0-35^0 (it is anticipated that units with ship-shaped hulls will have stability considerably beyond these ranges).

(ii) For column stabilized units, the area under the righting moment curve to the second intercept or the angle of downflooding, whichever is less, should be not less than 30 percent in excess of the area under the wind heeling moment curve to the same limiting angle.

(iii) For single buoy moorings, the units should have positive stability without taking into account the mooring system. Because of the possible variations in the design types, the reserve stability of each installation should be assessed for adequacy on an individaul basis. However, for units which are small or which are manned only at irregular intervals when a boat will be standing by, some relaxation may be allowed, e.g. the effect of the mooring system may be considered as assisting stability, where appropriate.

The static angle of heel due to wind should not exceed 15 degrees and the second intercept of the riighting moment and heeling moment curves should be at an angle of 30 degrees or more.

(e) Metacentric Height

For ship and barge-shaped units and bottom-supported units in the floating mode the metacentric height shall not be less than 0.5m. For column-stabilized units the metacentric height shall not be less than 1.0m for all conditions except while ballasting when it should not be less than 0.3m.

4.3.2 FLOATING DAMAGE STABILITY

A floating unit is to have sufficient stability and reserve buoyancy to remain stable and afloat when any two adjacent watertight compartments are flooded. It may be assumed that watertight closures to adjacent compartments are effectively closed and that damage penetration does not exceed 1.5 metres. Pipes capable of causing uncontrollable major flooding in other compartments should be located 1.5 metres or more from the outer skin.

The unit should have the ability to withstand the overturning moment of a wind of 25 meters per second (about 50 knots) from any direction without submerging any openings through which downflooding may occur, with the area under the righting moment curve to the second intercept or the angle of downflooding, whichever is less, should not be less than the area under the wind heeling moment curve to the same limiting angle.

The static angle of heel due to wind should not exceed 15 degrees.

4.3.2.1 Reserve Buoyancy for Column-Stabilized Units

The unit should be provided with sufficient buoyancy in the deck structure to remain afloat after the loss of buoyancy

provided by one column with the resulting waterline not less than 0.6m below any opening through which progressive downflooding may take place.

The righting arm should be positive for angles of heel 20 degrees beyond the above equilibrium condition with a maximum value not less than 1.0m.

4.3.3 TILT

There is some evidence that, in certain wave conditions, semi-submersibles with low values of metacentric height could develop steady angles of tilt about which they would oscillate. Reference 18 provides some information on this matter including calculation methods for assessing the possibility of 'tilt'. This matter should be examined in any new design that is likely to have a metacentric height sufficiently low that such tilt may result.

4.4 OFFSHORE LOADING UNITS

Such units which are normally unmanned should:-

(a) be provided where relevant with an anchoring system capable of withstanding safely:

i) a survival condition in accordance with the environmental criteria for its location;

ii) the limiting operation condition, to be considered as the worst environmental condition for operations, as stated by the owner, whilst the largest vessel to be used is still moored to the unit;

(b) have access for personnel (ladders, handrails, etc.) in accordance with the Guidance Notes;

(c) equipment on the unit to the satisfaction of the certifying authority;

(d) the design should take into account clauses of these guidance notes relevant to structural strength, stability, etc. as necessary.

4.5 FLARE STACKS

Normally unmanned structures used solely for flaring gas may be considered in the same manner as normally unmanned loading units.

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SECTION 5 SECONDARY STRUCTURE AND FITTINGS

SUMMARY

This section lays down requirements for secondary structures and fittings, giving general guidance and specific details as to the superstructure, the helicopter landing area, watertight compartments, protection against impact, living accommodation, decks, stairways, etc., fire protection, escape routes, hazardous areas, facilities for inspection and maintenance, and to the location and securing of modules and equipment. If the whole or a part of any of the secondary structures described below forms part of the primary structure, that part should comply also with the requirements of Section 4. Due care and attention should be paid to the pipeline risers which are complex and exposed parts of offshore installations and would, in the case of rupture, represent a danger to crew and environment.

5.1 GENERAL REQUIREMENTS

Each part of the installation should be considered in relation to the integrity of the structure generally. Reference should be made to the other sections of this guidance as necessary to determine the relevant considerations. Whether parts are initially built into the structure or are incorporated subsequently they should be subjected to the same scrutiny as to their individual characteristics and their relation to the whole structure.

At the early stages of design and layout of the installation, due consideration should be given to the location of the accommodation, processing, wellhead, drilling and other such areas, machinery spaces, helicopter landing decks, escapes and lifesaving equipment, having regard to the relevant hazards.

5.2 SUPERSTRUCTURE

5.2.1 STRENGTH, ETC.

Decks, deckhouses, skids, modules and other structures on decks should be capable of withstanding the loads and forces to which they will be subjected. Section 8.1.5 deals with the installation of equipment including cranes, fixed lifting appliances and derricks. These structures should be designed

in accordance with the recommendations in Canadian Standards Association CAN3-S16.1-M-78 Steel Structures for Buildings.

5.2.2 CLEARANCE

That part of the superstructure which is not designed to resist wave impact should have a clearance 'air gap' above the highest foreseeable wave crest superimposed on the maximum surge height over normal MHWS.

The clearance should be not less than 1500mm and may be achieved:

(a) in the case of a column-stabilized unit, by de-ballasting or by the response of the unit;
(b) in the case of a self-elevating unit, by elevating the hull, either in the first instance or when a storm is forecast. Any additional elevation must be within the designed limits of the unit;
(c) in the case of a fixed unit, by placing the platform at an appropriate level.

The 1500mm clearance is required in addition to allowances made for any settlement, subsidence and local increases in wave crest height caused by the submerged part of the structure or otherwise and for the accuracy with these and any other factors influencing the deck level are calculated.

When the structure may significantly modify the wave profile, this clearance should be demonstrated by a model test.

5.3 HELICOPTER LANDING AREA

5.3.1 DRAWINGS, CALCULATIONS, SPECIFICATIONS

General layout drawings of the helicopter deck should be prepared. The drawings should give the outlines of the helicopter deck including the surrounding areas. Distances to objects and structures outside the edge of the helicopter deck and their heights above the helicopter deck should be indicated. The location of the helicopter deck should be indicated. The location of the helicopter deck on the structure should be clearly evident from the drawings.

Strength calculations with material specifications should be prepared.

General lay-out drawings showing the helicopter deck structural members, indicating all main dimensions, should be prepared.

Detailed drawings showing the helicopter deck material, scantlings, supports and dimensions should be prepared.

5.3.2 ACCESSIBILITY

The helicopter deck should be located so that it is easily accessible from the crew accommodations and should be capable of being reached without passing through working areas. The deck should be located so as to provide clear approach and departure paths so that the largest helicopter intended to use the landing area can operate in all weather conditions which would allow the helicopter to fly.

5.3.3 LOCATION AND APPROACH SECTOR

The helicopter deck should have a take off and approach sector of at least 210 degrees free from obstructions. The base of the sector should be on the periphery of a reference circle with diameter "D" equal to the overall length of the helicopter for single main rotor helicopters, including rotors, and a reference rectangle of 0.9 and 0.75 times the measurement across the main rotors, in the fore and aft line, for tandem main rotors helicopters. Each leg of the sector should form an angle of minimum 105 degrees ± 15 degrees, with the reference circle radial or the reference rectangle bisector of the longer side at the base point.

Within the take off and approach sector there should be no obstructions protruding above the helicopter deck level. Exceptions may be made for singular obstructions imperative to the operation of the unit provided they do not rise more than 25 cm above the helicopter deck.

Within the remaining 150 degrees, ± 15 degrees, sector no obstruction should exceed a height in excess of 5 percent of the overall length of the critical helicopter (longest of the single or tandem rotor helicopters) within a distance of one third of the helicopters overall length from the periphery of the reference circle/rectangle. However, in order to safeguard access to stairways or installations of a similar purpose, a handrail of a height not exceeding 1m is permitted at the edge of the helicopter deck in this sector.

NOTES:

a) The location of a helicopter deck is often a compromise of various factors and it may be desirable to model test offshore structures aerodynamically. If it appears that the design parameters given in these guidelines cannot be achieved it may be necessary to impose restrictions upon helicopter operations as a result of such tests e.g. in relation to wind velocities.

b) It could be that the helicopter deck because of its location imposes severe restraints upon the helicopter operations because of wind velocities and directions. If such restraints are detrimental to helicopter operations it might be advisable to provide two separate decks which then should be located diametrically opposed to each other; each should comply with the regulations for such decks.

c) The helicopter deck should be located so that the clear approach and take-off sector is available, preferably into the prevailing winds, and is least affected by structure-induced turbulence or by high temperatures and turbulance from the exhausts of gas turbines.

d) Where gas turbines are installed and where the exhaust gases may affect helicopter operations, some form of exhaust plume indication, for example, by the production of coloured smoke, should be provided for use during helicopter operations. Unless it is obvious that the air temperature in the vicinity of the flight paths to and from the helicopter deck will not be affected by the exhaust plume, a survey of ambient temperatures should be conducted, during periods when the wind is blowing directly from the turbine exhaust ducts towards the helideck. Where the ambient temperature is increased by more than 2 or 3 degrees C the helicopter operator should be advised.

e) Helicopter decks which are located directly upon deep, slab-sided structures such as accommodation areas are liable to suffer from excessive vertical airflow components unless there is sufficient separation to allow airflow beneath the helideck. The combined effects of airflow direction and turbulence, prevailing wind, and turbine exhaust stack emissions, should be determined for each installation. This information should be made available to the helicopter operator but the vertical airflows resulting from winds of up to 25 meters per second generally should not exceed ± 0.9 meters per second over the landing area at the main rotor height.

f) Its may be necessary in difficult design cases for some form of permanent heat-sensor instrumentation to be installed to give guidance to the helicopter pilot on the temperature profile which may be encountered during operations of the structure.

g) It is desirable, particularly on fixed structures that in meeting the safety criteria details in these guidelines the helicopter deck should be located at or above the height of the highest point of the main structure, but in so doing it should be realized that if this entails a landing platform much in excess of 60m above sea level, the regularity of helicopter operations may be affected adversely in low base cloud conditions.

5.3.4 SIZE

Each helicopter deck should have a size equal to the overall length of the largest single main rotor helicopter that could be used on the structure. If tandem main rotor helicopters are used the deck size should be rectangular with the longitudinal length at least 0.9 D and the athwartship axis at least 0.75 D.

5.3.5 LOADS

The helicopter deck should be designed for the highest category (weight and size) of helicopter it is anticipated will use the deck. Loadings caused by wind, water, snow, ice, personnel freight, refuelling equipment and fire fighting equipment should be considered. After having considered all of the aforegoing possible loads the entire helicopter deck should nevertheless be designed to support a uniformly distributed deck load of 2 kN/m^2.

In addition to the 2 kN/m^2 deck load the helicopter deck should also be designed to carry a load equal to the helicopter wheel loadings of maximum take off weight plus inertial forces from the helicopter due to anticipated motions of the drilling or production unit, in the case of floating structures, and applicable environmental loadings. For the girders, stanchions, truss supports, etc., the structural weight of the helicopter deck should also be considered and also the self weight of supporting members, where appropriate.

The helicopter deck should also be capable of supporting loads from the helicopter landing gear applied anywhere on the helicopter deck area. Each of these loads, in addition to the uniformly distributed load of 2 kN/m^2 should be 75 percent of the maximum take off weight of the helicopter acting

simultaneously on two points $0.1m^2$ in area and 4m apart, or should be the manufacturer's recommended wheel loading in accordance with the disposition and nature of the main landing gear of the helicopter. Should the upper deck of a superstructure or deck house be used as a helicopter deck and the spaces below manned (quarters, bridge, control room, etc.) then the impact loading should be multiplied by a factor of 1.15.

Due consideration should be given to the sympathetic response of the helicopter deck. The dynamic load due to impact on landing should be increased by a structural response factor dependent upon the natural frequency of the deck slab when considering the design of supporting beams and columns. This increase in loading will usually only apply to slabs with one or more freely supported edges. A minimum structural response factor of 1.3 should be used.

The supports of the helicopter deck should be designed also to resist a horizontal lateral point load equivalent to 0.5 of the maximum take off weight of the helicopter, together with the expected wind loading, applied in the direction which will produce the most severe loading condition.

The punching shear load of the helicopter under carriage, on the helideck, should be checked. Close attention should be given to detailing at the junction of the supports and the deck.

The loads mentioned in the foregoing paragraphs, should be considered in combination with the structure dead load and the wind forces.

5.3.6 DECK SURFACE

The helicopter deck surface should be in a condition or so equipped that the coefficient of friction, even when wet, is at least 0.4. The helicopter deck should be provided with tie down hooks and a rope net to secure a parked helicopter and to prevent sliding.

5.3.7 SLOPE

The helicopter deck should have a slope of 1 in 100 to avoid the accumulation of water and should have drainage facilities that prevent the collection of liquids and prevent liquids from spreading to or falling on other parts of the unit, having regard to the use of fire fighting equipment and the possible spillage of fuel.

5.3.8 SAFETY NET

The helicopter deck should be surrounded by a safety net, at least 1m wide and of sufficient strength to support a weight of at least 200kg per meter. The outer edge of the net should not extend beyond 5cm above the level of the deck.

5.3.9 ACCESS

The helicopter deck should be made accessible by two widely separated gangways.

5.3.10 MARKINGS AND LIGHTS COLOURS

5.3.10.1 Colours

The helicopter deck should be grey or green in colour. Areas not intended for landing should be clearly designated in red. The take-off/landing area markings should consist of a painted circle 40cm in width and 6 m in diameter enclosing the center of the take-off/landing area. The perimeter of the helicopter deck should be indicated by a 40cm wide strip. The colour of the markings should be of maximum contrast to the colour of the take-off/landing area.

5.3.10.2 Marking of Obstructions

Obstructions located closer to the helicopter deck than 1 rotor diameter and/or representing a hazard to flight should be painted in alternating black and yellow stripes and, if necessary, furnished with red warning lights of a minimum power of 10 candelas.

5.3.10.3 Warning and Deck Lights

The top of the drilling tower, vent stacks, radio towers and cranes should each be equipped with a red warning light, visible from all directions and having a minimum power of 50 candelas. The helicopter deck should be furnished with lights along the edge and such lights should consist of alternating blue and yellow colours not more than 3 meters apart. The light should have a power of 25 candelas. Lights should be connected to the emergency power circuit for automatic activation in the event of a failure of the main power source.

5.3.10.4 Floodlights

The landing area should be floodlit if intended for night use. The floodlighting should be so arranged as not to dazzle the pilot and, if elevated and located off the landing area, the system should not present a hazard to helicopters landing or taking off. It is particularly important to confine the illumination to the landing area since any light overspill could cause reflections from the sea. The floodlighting should be controlled from the radio room or helicopter landing officer's office.

5.3.11 DRAINS

The helicopter deck is to be encircled by a drain or similar installation for the purpose of collecting any leaking fuel and to provide for its discharge over the side. Construction should be such as to make it capable of withstanding deleterious fuels.

5.3.12 COAMINGS

There should be a safety shoulder of 5cm high along the edges of the deck in order to prevent personnel or cargo from slipping overboard. The safety shoulder should be so constructed as to permit ready discharge of fuel and water. Adequate drains must be provided for the disposal of liquids from the helicopter deck. Construction should be such so as to make it capable of withstanding burning fuel.

5.3.13 WIND DIRECTION INDICATOR

A wind direction indicator cone should be located adjacent to the helicopter deck so as to be conspicuous from all directions and indicate accurate wind direction but not be a hazard to flight. It should be lighted for night operations.

5.3.14 HELIDECK BEACON

Where it is considered necessary to provide a beacon it should display a green/yellow/white signal. It should rotate clockwise as viewed from below at a speed which will produce a flash rate of 30 to 60 flashes per minute.

5.3.15 SIGNS

Signs should be posted at conspicuous locations near the helicopter deck prohibiting -

The presence of personnel on the helicopter deck during take-off and landing.

Smoking on the helicopter deck whenever helicopter refuelling operations are in progress or when the helicopter is parked.

Personnel to move around the tail rotor during embarkation and disembarkation.

5.3.16 REFUELLING FACILITIES

Helicopter landing areas equipped with permanent fuelling and storage facilities should have those facilities suitably isolated from enclosed spaces or other areas which contain a source of vapor ignition.

The fuel should be held in suitable permanent storage tanks which are to be firmly secured to the deck and suitably insulated against mechanical damage and excessive temperatures. They should be positioned in a location and fitted with a device that allows the tank to be released overboard in an emergency and the control for the release device shall be at least 10 metres from the tanks.

Storage and pumping facilities should be electrically grounded and closed circuited and all pump motors should be totally enclosed and explosion proof.

Trigger type nozzles should be used on fuel hoses to prevent spillage when actual fuel operations are not taking place.

Permanent coamings should be erected around the storage and pumping facility complete with a suitable drainage system to contain any spillage of fuel or any applied fire extinguishing medium.

5.3.17 FIRE PROTECTION FOR HELICOPTER FACILITIES

The helicopter deck should be provided with at least two fire hydrants, one at each side, and for each hydrant a fire hose with an approved fog nozzle or applicator suitable for spraying water on fuel fires.

The helicopter deck should have sufficient foam agent to discharge foam continuously for at least 5 minutes at maximum discharge rate and have at least enough foam agent to cover an area equivalent to the swept rotor area of the largest helicopter for which the deck is designed with foam at:

a) if protein foam is used, 6 liters per minute for each square meter of area covered for 5 minutes.

b) if aqueous film forming foam is used, 4 liters per minute for each square meter of area covered for 5 minutes, and

There should be equipment available capable of discharging from each hose at 700 kPa pressure;

a) a single foam stream at a rate of at least 340 liters per minute, and

b) a foam spray at a rate of at least 190 liters per minute.

Each system should have at least one hose which should be reel mounted and be long enough to reach any point on the helicopter deck. Each hose that discharges foam should have a nozzle that has foam stream, foam spray and off positions.

Portable dry powder extinguishers of a total capacity of not less than 45kg, portable carbon dioxide extinguishers of a total capacity of not less than 18kg and portable foam extinguishers of a total capacity not less than 250 litres should be provided and stored near the means of access to the helicopter deck. The CO_2 extinguisher should be furnished with an extension hose nozzle of a length sufficient to reach the engine of the helicopter in use.

There should be a fixed dry chemical fire extinguishing system consisting of at least the following:

(a) one dry chemical generator;

(b) two dry chemical trigger nozzles, each of which has a capacity of not less than 2 kilograms per second;

(c) two dry powder bases each of which is not less than 30 metres long;

(d) 250 kilograms of dry chemical or 1.5 kilograms of dry chemical for each square metre of helicopter deck area, whichever is the greater.

5.3.18 FIRE PROTECTION FOR HELICOPTER FUELLING FACILITIES

Each helicopter fuelling facility should have a fire protection system that discharges one of the following agents in the amounts prescribed for the agents over the area of the fuel containment systems around the fuel tanks, fuel transfer pumps and fuel nose reels:

a) protein foam at the rate of .7 liters per minute for each square meter of area covered for 5 minutes,

b) aqueous film forming foam at the rate of 4 liters per minute for each square meter of area covered for 5 minutes,

c) 22.5 kg of dry chemical (B.4 semi-portable) for each fuelling facility of up to 25 square meters.

If the fire protection system described in 5.3.17 is designed so that it covers both helicopter deck as well as the fuelling facilities then the system should have the quantity of agents required by this section as well as those of 5.3.17.

A fuel pump shut-off should be installed at each of the access routes leading up to the helicopter deck.

All firefighting equipment should be kept in good working order and ready for use at all times.

5.3.19 SPARE SUPPLY

In addition to the requirements of 5.3.17 and 5.3.18 the drilling platform or production facility should at all times keep a reserve supply of foam liquid and powder equivalent to 200 percent of the requirements.

5.3.20 EMERGENCY EQUIPMENT

The following minimum equipment should be placed on or near the helicopter deck -

2	-	crash axes
2	-	crash knives
1	-	pair 24 inch bolt cutters
1	-	hack saw with 2 spare blades
1	-	hammer
1	-	crowbar
1	-	plate scissors

2	-	hand lanterns
2	-	breathing apparatus
2	-	non-conducting boots and gloves
2	-	helmets
2	-	protective clothings
2	-	life lines
1	-	pair of slipjoint pliers

This equipment should be kept so that it is easily available, preferably in a lockable locker/box with a breakable glass/plastic window installed. The locker/box is to be red and to be clearly marked "EMERGENCY EQUIPMENT". A three meter metal boat hook should be kept handy in a suitable place in the vicinity of the emergency equipment locker/box.

5.3.21 PERSON IN CHARGE

One individual should be made responsible for operations conducted on the helicopter deck and it will be his responsibility to assure -

That instructions are provided insuring that the helicopter deck is clear of personnel, loose objects, and that the rope net is properly secured, etc. prior to landings and take-offs, that all firefighting equipment is in a proper position,

That the personnel concerned have been given instructions and training in rescue and firefighting procedures, and

That passengers are provided with guidance during embarkation and disembarkation.

5.4 WATERTIGHT COMPARTMENTS

5.4.1 STRENGTH OF WATERTIGHT COMPARTMENTS

The construction of a mobile installation should be such that watertight compartments exist in those parts of the installation that are buoyant in the floating condition. In the case of large hollow members subject to immersion there should be at least one watertight subdivision. Whether a compartment is bound by watertight decks, bulkheads or other structures, the whole should be capable of withstanding the hydrostatic pressures likely to be imposed during service. Compartments should be such that the installation meets the stability requirements contained in Section 4. Their design and construction should be of a standard not less than that required for ocean-going cargo ships.

5.4.2 OPENINGS IN WATERTIGHT COMPARTMENTS

All openings in watertight compartments should have an effective means of closure as appropriate and should be formed and stiffened so that the hole, when closed, has a strength equivalent to the unpierced deck, bulkhead or other structure. Doors, hatches and other closing appliances should be made of suitable material, be efficiently constructed and be permanently and strongly attached to the bulkheads or other surrounds; doors should be fitted with gaskets and positive means of securing them from either side. Each discharge with an inboard opening should have an automatic non-return valve and either a second such valve or a device for closing the opening from a position outside and above the compartment. Each inlet should be fitted with a closing device which can be operated from a readily accessible position. Pipes and ducts led through compartments should be constructed so as to maintain the watertight integrity of the compartment. Door sills and hatch coamings in open decks should be of sufficient height to prevent the ingress of water. Detailed construction of all openings and their means of closure should follow ship classification practice. The control room should have a panel to continuously monitor which watertight doors or hatches are opened or closed.

5.4.3 OPERATIONS MANUAL - OPENINGS AND MEANS OF CLOSURE

The Operations Manual should contain particulars of all openings and their means of closure, with guidance as to the circumstances when closure sould be needed, and instructions on the appropriate action to be taken to close and to open each opening.

5.5 PROTECTION AGAINST IMPACT

Special attention should be given to the adequacy of fendering in relation to movements of supply vessels, and, in ice-infested waters, of sea ice and 'bergy bits', in the light of the forces they may exert on the installation. Fendering should be capable of absorbing the energy of the impact of these objects. If it is designed for partial collapse in the event of overload, it should be so constructed as not to be capable of becoming detached from the main structure in a manner which could damage either the installation or the vessel striking it. Its depth should be such that a vessel could not become trapped underneath it, particularly at low tide. Energy absorption mechansims could be built in and adequate stability provided against progressive collapse. Design should include

the effect of an impact of a ship of 5000 tonnes displacement coming into contact at a velocity in meters per second equal to one-half the maximum significant wave height in which they are allowed to operate. It should be assumed that the absorbtion of kinetic energy is divided between the fendering/installation and the impacting object relative to their strength and stiffness.

Conductors, risers and pipelines carrying petroleum products under high pressure should be so situated as to minimize the possibility of accidental damage by objects impacting the installation.

5.6 LIVING ACCOMMODATION

5.6.1 GENERAL

All living accommodation should provide adequate protection from the weather, be properly heated or cooled as necessary; be located outside any hazardous areas and separated as far as practicable from machinery so that noise and vibration will not have an adverse effect on the health of those on board the structure. Thermal insulation should be incorporated as necessary.

5.6.2 SLEEPING ACCOMODATION

There should be sufficient beds or bunks for the number of persons expected on board the installation without any sharing by persons on successive shifts. On production installations it is recommended that accommodation, for permanent workers, be provided on the basis of individual rooms. Under other conditions any room designated as sleeping accommodation should be used by no more than two persons except for drilling camps, exploration units and short-stay maintenance workers' accommodation when four persons per room may be accepted subject to the agreement of the certifying authority.

The minimum floor area provided for each person in a room designated as sleeping accommodation should be not less than 3 square metres. There should be at least 900mm distance between each bed or bunk, or fixed or moveable partitions providing privacy for each person. No bunk should be more than two-tier; the distance between the bottom and top tier of a bunk and the top tier and the ceiling should not be less than 900mm. There should be adequate space for the occupents to hang their clothes and there should be one lockable drawer for each person

in the room. If women are to be employed on the installation separate sleeping accommodation should be provided for them.

5.6.3 TOILET AND WASH-ROOMS

There should be at least one basin with a hot and cold water supply for every six persons, and a water closet (not a urinal) and a bath or a shower for every eight persons. At least one bath should be available at all times for first-aid purposes. If women are to be employed on the installation, separate facilities should be provided for them.

5.6.4 CLOTHES DRYING AND LAUNDRY ROOMS

There should be a clothes drying room having independently controlled heating and ventilation. It should be fitted with racks and rods allowing adequate space for drying of clothes having regard to the the number of persons likely to be on board the installation. There should be a separate room containing laundry facilities; this should be in addition to the personal wash-room facilities referred to above.

5.6.5 MESS ROOM AND GALLEY

There should be a mess room and galley (together with food and water storage facilities) of adequate capacity to cater at one time for at least half the number of persons likely to be on board the installation.

5.6.6 RECREATION ROOM

There should be a recreation room large enough to provide seating at one time for at least half the number of persons likely to be on board the installation.

5.6.7 FIRST-AID WARD

There should be either a first-aid or medical ward depending upon the number of persons likely to be on board the installation.

5.7 DECKS, STAIRWAYS, ETC.

5.7.1 SURFACES AND CONSTRUCTION

Deck surfaces should incorporate non-slip surfaces where practicable of sufficient strength to withstand local loading without permanent deformation. Dimensions and methods of construction of corridors, walkways, stairways and ladders should be at least the standard required on sea-going cargo ships.

5.7.2 EXPOSED PARTS AND OPENINGS

The edge of every floor, gangway and stairway, and all openings down which a person could fall more than 2 metres, or into the sea, should where practicable be provided with a toe board not less than 150 mm high and suitable guard rails of adequate strength comprising at least three courses so arranged that the lowest rail is not more than 150 mm above the toe board and the highest rail is not less than 1 metre above the floor, gangway or stairway and the openings between the rails are not more than 400 mm; wire or wire mesh of sufficient strength may be substituted for guard rails below the top rail.

5.7.3 LADDERS

All ladders over 6m in height (including those incorporated in equipment) and not intended for use solely in cases of emergency, should be protected either by safety cages or if this is not possible by efficient safety harness devices. Where provided, safety cages should extend from 2.5 m above the bottom of the ladder. Where practicable, rest platforms should be provided at not more than 9m intervals.

5.8 FIRE PROTECTION FOR ACCOMMODATION SPACES AND CONTROL STATIONS

5.8.1 MATERIALS AND TESTS

(a) Materials required to provide structural fire protection should comply with the requirements of a relevant code or standard. The materials and their method of assembly should be tested at a recognized independent establishment. In general the requirements are to be those of Regulation 3 of Chapter II-2 of the International Convention for the Safety of Life at Sea, 1974 (SOLAS), or with other equivalent

international or national standards. Users should take great care when interpreting the results of tests other than those well known to them.

(b) A non-combustible material is one which is non-combustible in accordance with SOLAS Regulation 3(a) of Chapter II-2.

(c) A standard fire test may be the test in ASTM: E119-76, or in SOLAS Regulation 3 (b) of Chapter II-2.

(d) 'A-60' Class Divisions are those divisions formed by bulkheads and decks which comply with the following:

(i) They shall be constructed of steel or other equivalent material.
(ii) They shall be suitably stiffened.
(iii) They shall be so constructed as to be capable of preventing the passage of smoke and flame up to the end of the one-hour standard fire test.
(iv) They shall be so insulated that, if either face is exposed to the standard fire test for one hour, the average temperature on the unexposed face will not increase at any time during the test by more than 139°C above the initial temperature nor shall the temperature at any point on the face, including any joint, rise more than 180°C above the initial temperature within 60 minutes.
(v) All materials entering into the construction and erection of 'A-60' class divisions shall be of non-combustible material.

(e) 'B-15' class divisions are those divisions formed by bulkheads, ceilings and linings which comply with the following:

(i) they shall be so constructed as to be capable of preventing the passage of flame to the end of the first 30 minutes of a standard fire test;

(ii) they shall be of such material that if either face is exposed to the first 30 minutes period of a standard fire test, the average temperature on the unexposed face will not increase at any time during the first 15 minutes of the test by more than 139°C above that initial temperature on that face, nor shall the temperature at any point, including any joint, rise more than 225°C above the initial temperature;

(iii) all materials entering into the construction and erection of 'B-15' class division shall be non-combustible.

(f) Continuous 'B-15' class ceilings are those 'B-15' class ceilings which terminate only at an 'A-60' class or

'B-15' class division or the external boundaries of the accommodation block.

(g) 'Low spread of flame' means a spread of flame classification of not more than 2 according to BS 476, Part 7.

(h) 'Equivalent material'; where these words are used in the expression 'steel or other equivalent material' they mean any material which, by itself or due to insulation provided, has structural and integrity properties equivalent to steel at the end of a one-hour fire test.

(i) 'Control stations' are those spaces in which the radio, main navigating, central fire detection or control, the installation central internal communication equipment, the emergency source of power or the emergency switchboard is located.

(j) 'Accommodation spaces' are those used for dining rooms, recreation rooms and similar permanently enclosed spaces and corridors, lavatories, cabins, offices, sick-bays, living quarters, pantries and lockers and similar spaces.

5.8.2 STRUCTURAL FIRE PROTECTION

(a) All materials shall be non-combustible except that consideration my be given to the use of combustible material where it can be demonstrated to the satisfaction of the certifying authority that any other required property or use of the materials precludes those materials from being non-combustible.

(b) Every deck and its supporting structure within the accommodation area, which is not required to be an 'A-60' class division, should be constructed of material which by itself or due to installation provided will not lose its structural stability and the fire integrity of the deck when subjected to a 60-minute standard fire test.

(c) All corridor bulkheads, where not required to be 'A-60' class division, should be 'B-15' class divisions extending from deck to deck. Where continuous 'B-15' class ceilings are fitted the bulkheads may terminate at the continuous ceilings. 'B-15' class doors should be fitted in 'B-15' class bulkheads and the doors to cabins and public spaces in such bulkheads may have a louvre in the lower half. Ventilation openings or louvres should not be provided in doors of divisions forming stairway enclosures.

(d) (i) Openings in all bulkheads and decks in the accommodation spaces should have permanently attached means of closing which will maintain the fire integrity of such bulkheads and decks.

(ii) Where 'A-60' class or 'B-15' class divisions are pierced for the passage of electric cables, pipes, trunks, structural members or for other purposes, the arrangements should be such to ensure that the fire resistance is not impaired.

(iii) Air spaces enclosed behind ceiling, panellings or linings should be divided by close-fitting draught stops spaced not more than 14 metres apart. Draught stops should be fitted transversely if the length of the space exceeds 14 metres and lengthwise if the breadth exceeds that dimension.

(e) Internal stairways, ladders and crew lift trunks within the accommodation areas should be constructed of steel or equivalent materials. Stairways serving more than two decks should be enclosed within a trunk constructed of 'A-60' class divisions and shall have self-closing doors. Stairways connecting only two decks need only be required to be enclosed at one level by divisions having the same fire integrity and structural stability as the deck and having self-closing doors. Stairway enclosures should be in direct communication with the escape corridors.

(f) All boundaries of control stations should be 'A-60' class divisions.

(g) (i) Boundaries separating any machinery space, or storeroom containing paint, oil, gases or other inflammable materials, from any galley or accommodation area, should be 'A-60' class divisions. Boundaries separating these spaces from each other should be of non-combustible material which will not lose its fire integrity after being subjected to a standard fire test for one hour. The surface of insulating materials fitted to the inside of bulkheads and decks forming the casings and crowns of compartments containing or affected by oil and oil vapour should be impervious to oil and oil vapour.

(ii) Boundaries separating accommodation spaces from any galley or any pantry containing food cooking appliances should be constructed of non-combustible material which by itself or due to insulation provided will not lose its fire integrity when subjected to a 60-minute standard fire test. Doors and the shutters to serving hatches should be so constructed that the intergrity of the bulkheads is maintained and should be capable of being readily closed from a position outside the galley or pantry.

(iii) Where galley supply or exhaust ventilator trunking pases through accommodation or any other enclosed spaces the trunking within those spaces should be of steel covered with a

fire-resisting insulation material of a type and thickness acceptable for an 'A-60' class division.

(h) Bulkheads separating the accommodation area from wellhead or process areas should be 'A-60' class divisions. Alternatively other equivalent fire protection may be provided between the accommodation and these areas.

(i) Primary deck coverings should be of a type which will not readily ignite and should be suitable for the intended service.

(j) Paints, veneers and other surface finishes used on surfaces in concealed or inaccessible spaces and on exposed surfaces (except furniture, furnishings and floor coverings) should be of low flame spread.

(k) Pipes intended to carry oil or other combustible fluids or gasses (other than those concerned with the primary process of the installation) should be of a material and construction acceptable to the certifying authority.

(l) Overboard scuppers, sanitary discharges or other outlets close to the water should be of a material unlikely to fail in the event of fire and thereby give rise to the danger of flooding.

(m) Materials such as organic foam insulation which may give off large quantities of toxic fumes or smoke should not be used. Insulation for cold store/refrigerated spaces are usually of the foam type which is combustible; however, this is by far the best insulant for these spaces. Consequently combustible foam insulation may be accepted for this use provided:

(i) the foam is of a fire retardent type;
(ii) the foam is totally enclosed in stainless steel or other such corrosion resistant material having all joints suitably sealed, e.g. welded or brazed;
(iii) the insulation and its casing shall not form any part of the accommodation structure, i.e. decks, bulkhead, etc.
(iv) a suitable entry shall be made in the Operations Manual regarding the fitting and position of this insulation and precautions to be taken during any repairs.

5.9 NOISE AND VIBRATION

This section describes the way in which noise and vibration, as affecting human exposure, should be taken into account in the design and construction of offshore installations. It details recommended maximum levels for all areas of the installation

and provides basic guidelines to be followed when designing and laying out an installation to minimize potential noise and vibration problems.

This section does not attempt to give a comprehensive design guide since each installation will clearly present particular problems, depending upon its size and function. It is important that consideration of noise and vibration should form an integral part of the platform design from the inception of the project, when, for instance, major changes in platform layout can be made, to reduce potential problems. Where necessary, specialist advice should be obtained. For acoustic design purposes all calculations should be performed in the eight octave bands centred between 63 Hz and 8 kHz. The noise rating curve (references 4 and 5), numerically 5 dB less than the dBA criterion, should be used as an approximate limit for the frequency spectrum of the noise. The overriding requirement, however, is that the dBA level should be met.

Noise levels should be limited throughout the installation in order to:

(a) minimize the risk of hearing damage to personnel in work areas;
(b) ensure that warning signals are audible;
(c) allow adequate speech, telephone and radio communication;
(d) maintain working efficiency;
(e) provide an acceptable sleep and recreation environment in accommodation areas.

In a similar manner vibration levels should be limited in order to:

(a) prevent a health hazard to personnel;
(b) maintain proficiency of personnel in performing designated tasks;
(c) provide an acceptable sleep and recreation environment in accommodation areas.

5.9.1 NOISE

5.9.1.1 Standards and Definitions

5.9.1.1.1 Terminology

Interpretation of the terminology used in these recommendations should be in accordance with reference 6. All noise limits in

this section are quoted as 'A' weighted sound pressure levels (dBA). In all cases the reference sound pressure is 2 X $10^{-5} N/m^2$.

5.9.1.1.2 Application of Criteria

The noise criteria should be met in full at any spatial location within a designated area to which personnel may have access, except for work areas and other areas housing noisy equipment where, for design purposes, the recommended criteria should generally apply at a minimum distance of 1 metre from operational equipment. If personnel are normally required to work closer than a distance of 1m to equipment then the noise criteria should also apply at these locations.

5.9.1.2 Criteria

5.9.1.2.1 Maximum Permissible Noise Levels (Overriding Limits)

Notwithstanding the criteria specified for the designated areas, under no operational design condition should the unprotected ear be exposed on a continuous or intermittent basis to sound pressure levels exceeding 115 dBA.

5.9.1.2.2 General Work Area Noise Limits

All areas, other than those stated in Sections 5.9.1.2.3 and 5.9.1.2.4 should meet the general work area noise level limit of 83 dBA for a 12-hour working day. This limit is based on hearing damage risk considerations and is derived from reference 1. If shift lengths are less than 12 hours then the noise limit may be increased in accordance with this code of practice, i.e. a noise limit of 85 dBA will apply for an eight-hour shift. These limits generally apply for broad band noise. Where a noise exhibits dominant tonal characteristics, then it is desirable to supress such characteristics (reference 2).

5.9.1.2.3 Specific Work Area Noise Limits

Where reliable speech, telephone or radio communication is required, or demanding mental tasks must be performed, then the noise limits for these areas should be considerably less than the limit for general work areas (reference 3). Any tonal characteristic should be suppressed so as not to give rise to annoyance. These limits refer to background noise, including

ventilation and external noise sources, but not to manually controlled operations involving inherently noisy equipment, e.g. drilling, mixing, etc., for which general work area noise limits shall apply. This does not include the radio/communications room in which noisy office equipment, e.g. telex machines, should either not be installed or should be suitably quietened.

5.9.1.2.4 Sleeping/Reaction Area Noise Limits

The noise should be limited such that satisfactory recreation, rest and sleeping conditions are required. Any tonal characteristics should be suppressed so as not to give rise to annoyance.

5.9.2 VIBRATION

The vibration should be limited to the acceptability of exposure of human beings to vibration and should be in accordance with reference 7.

5.9.3 GUIDELINES TO MINIMISE NOISE AND VIBRATION PROBLEMS

5.9.3.1 Equipment Selection

The need for noise and vibration control should be taken into account when deciding which of different production methods or processes are to be used.

Noise limits for individual items of machinery should be included as part of the general equipment specification issued to vendors. These individual limits should be set taking into account the location of all equipment in a particular area and its acoustic environment such that, in total, the relevant area noise should be within acceptable limits. In certain cases the installation of sound absorptive materials in work areas may allow less stringent equipment noise specifications to be set. All vendors should supply documentary evidence of the noise levels expected to be created by their equipment. Noise tests may conveniently be included as part of the normal mechanical acceptance testing of equipment.

In similar manner vendors should supply all available vibration test data for their equipment, together with mounting and foundation details.

5.9.3.2 Noise Control Measures

Noise control equipment should be designed to cause minimum interference with routine inspection and maintenance of equipment. Suitable access doors and observation windows should be provided and where necessary the control measure should be easily removable.

Note that personal hearing protectior should not be used as a substitute for effective noise control.

5.9.3.3 Vibration Control Measures

The selection of suitable isolators where fitted should take into account structural design loads and environmental conditions.

5.9.4 OTHER CONSIDERATIONS

5.9.4.1 Fog Horns

Consideration should be given to the choice and location of fog horns in order to minimise potential annoyance to personnel within accommodation areas.

5.9.4.2 Warning Signals

The amplitude and frequency of audible warning signals should be chosen in order that they are effective under all operating conditions where possible. Other methods such as visual warnings should be considered if necessary. Public address systems should be designed to provide the necessary speech intelligibility requirements in all areas.

5.10 ESCAPE ROUTES

All areas likely to be regularly manned should have to least two well-marked escape routes situated as far apart as practicable and leading to abandonment stations on the helicopter deck, on the survival craft embarkation deck, other open decks, or at sea level. All working areas, large accommodation rooms and any corridors more than 5 metres long should possess at least two exits which lead to escape routes. No escape routes should be obstructed by equipment or other fixed item, and all exit doors along the route should be readily openable. A plan of escape routes should be included in the Operational Manual.

5.11 HAZARDOUS AREAS

The installation should be classified into hazardous and non-hazardous areas in accordance with a recognised standard or Code of Practice. A plan or plans should be prepared showing these areas, and a copy of the plan should be included in the Operations Manual.

NOTES

1. The techniques of area classification have been developed for the selection of electrical equipment. However, provided all influencing factors are taken into consideration these techiques may be applied to other technologies.

2. During drilling and workover operations electrical apparatus within 15 metres of the wellhead (rotary table) or attached to the derrick which has to operate under conditions of breakdown of the measures of control (such as blowout) should be suitable for a hazardous area.

5.12 FACILITIES FOR INSPECTION AND MAINTENANCE

The design should incorporate features to facilitate the periodic inspections required by the regulations and such additional inspections as may be required from time to time. These features should include spaces or decks reserved for, or readily available for, diving equipment and plant, measures to facilitate the identification of members or location; and other measures to facilitate the work of divers and to reduce the hazards to which they may be exposed. Similar provision should also be made to facilitate maintenance works, to such extent as they may reasonable be foreseen. The arrangements and function of these facilities should be described in the Operations Manual.

5.13 REMOVAL, DAMAGE OR ABANDONMENT

The Minister will invariably attach as a condition of his consent to the erection or placing of a structure in a fixed postion, a requirement that if the structure is later removed or abandoned, the site will be cleared to his satisfaction, and if the structure is damaged or wrecked, or the whole or any part of it goes adrift, such steps (including if he considers it necessary the removal of all or any part of the structure) will be taken, within such period as he directs, to prevent danger or obstruction to navigation.

The degree of removal or clearance will depend on the location and circumstances of each individual structure, and it is not possible to cover every contingency. However, the general line followed is that:

(a) The Minister's requirements for site clearance will be dictated solely by considerations of navigational safety and obstructions to fishing.

(b) Any moored installation (e.g. an SPM3) will have to be entirely removed should it become redundant; but in deep water removal of the actual moorings on the sea-bed may not be required.

(c) A permanent structure standing on the sea-bed will be allowed to remain in situ when it becomes disused, subject to review and provided that the navigational markings specified in the consent remain in being and the structure is maintained or repaired to the extent that no deterioration or disintegration occurs which could result in danger or obstruction to navigation.

(d) In the event that (c) cannot be compilied with, the whole structure will have to be dismantled or removed to a depth below mean sea level which will be specified by the Minister. This depth will be such that the portions of structure which remain will be no danger to any vessel of a draught capable of being navigated in the waters concerned; in coastal waters this requirement could of course entail complete clearance to sea-bed level.

(e) The criteria in (d) will also apply to sub-sea completions and similar under-water installations.

(f) The achievement of the specified clearance will have to be supported by an independent survey. Assurance will also be required that there will be no possibility of any part of the remaining structure breaking off in the future and floating on the surface.

(g) In exceptional circumstances, where there is a need to safeguard submarine navigation, a structure in deep water may have to be removed to a much greater depth below sea level than would be dictated by the needs of surface shipping.

5.13.1 BREAK-OUT AND RETRIEVAL OF GRAVITY INSTALLATIONS

Where the structure is to be lifted and relocated during service, the forces imposed on the structure during break-out from the foundation should be determined. Means should be

provided to check or brake the rapid rise of the structure once it has lifted from its foundation.

Break-out may be achieved by first reducing ballast to attain neutral buoyancy, then pumping water in below the base slab in order to form a gap. An over-pressure corresponding to the break-out force will eventually develop. Only when the break-out is complete should ballast be further reduced in order to float the structure. Means of pumping water under the base slab should be incorporated in the initial design if retrieval or relocation is considered.

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4. ISO Recommendation R-1996, 'Assessment of Noise with Respect to Community Response'.
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SECTION 6 MATERIALS

SUMMARY

This section sets out the requirements for materials used in the construction of installations, and gives specific guidance on the selection and testing of the main materials in use and the codes and standards that are available.

6.1 GENERAL REQUIREMENTS RELATING TO MATERIALS

6.1.1 SUITABILITY

All materials should be suitable for the service conditions of those elements of the installation in which they are used. Account should be taken of the loads imposed on those elements including those caused by the environment, the resulting stresses, deformation and possible causes of failure. Non-combustible materials shall be used except where any required property or use of material precludes non-combustibility. Materials, other than those specified on the design drawings, including materials of a higher strength grade, should not be substituted in any member designated as being critical to the strength or integrity of the structure without the approval of the owner and the certifying authority. For all materials, appropriate testing should be carried out in accordance with relevant standards or Codes of Practice and the certified reports of those tests made available for scrutiny.

6.1.2 CORROSION PROTECTION

6.1.2.1 General

All materials should have appropriate resistance to corrosion and should be chosen to avoid electrolytic corrosion problems. Alternatively materials should be adequately protected from the effects of corrosion. It should be noted that the general corrosion rate in cold sea water is enhanced as compared to warmer waters and any protection should be designed accordingly. When considering the degree of surface protection necessary for an offshore installation, the structure should be divided into zones or areas as defined below and corrosion protection afforded in accordance with 6.1.2.3 below.

6.1.2.2 Zone Definitions

The degree of exposure of an offshore installation for corrosion protection should be considered in three external zones and internal zones as defined below:

Splash zone: The splash zone is defined as that part of the structure ranging between 65 percent of the 100 year (average) wave height superimposed on the highest astronomical tide and 35 percent of this wave height below the lowest astronomical tide.

Submerged zone: The region below the splash zone including buried parts.

Atmospheric zone: The region above the splash zone.

Internal zones: Regions of the structure which are not externally exposed. Internal zones are to be defined for each structure.

6.1.2.3 Zone Protection

Steel structures should be treated according to zone as follows (reference 1):

(i) The splash zone should be protected by one or more of the following methods:

extra steel (at least 12 mm) in excess of that needed for strength; or
suitable wraps/sheathing with the annulus completely sealed by welding; or
suitable coatings.

Normal practice would consist of both a protective coating and a corrosion allowance. The effectiveness of the method should be well documented for the rigorous duties required.

(ii) The atmospheric zone should be protected by proper surface preparation and use of selected materials: various paint systems and galvanizing have proved satisfactory; application procedures should follow the manufacturer's recommendations.

(iii) The external submerged zones should be protected by cathodic protection with sacrificial anodes or other suitable means. Anodes should be securely fixed and installed in such a way as to prevent their being damaged by craft coming alongside the installation. The internal submerged zones should be

protected by sacrificial anodes and/or by suitable surface paint coating.

Reference should be made to the National Association of Corrosion Engineers Standard RP-01-76, 'Control of Corrosion on Steel, Fixed Offshore Platforms Associated with Petroleum Production'.

Concrete structures should be protected by compliance with the requirements for cover, crack limits, etc., in Section 4 and other requirements of this section.

6.1.2.4 Operations Manual - Maintenance

The Operations Manual should contain particulars of the materials used and the methods employed to limit corrosion or other deterioration. Guidance should be given on the correct maintenance procedures.

6.2 STRUCTURAL STEEL

6.2.1 GENERAL

The guidance in the paragraphs below relate primarily to steels described in the Canadian Standards Association Code G40.21-M1978 and G40.20-M1978. Where other steels are used they should have properties suitable for their intended service. Existing installations in which materials do not comply with the minimum values given below may be accepted for certification depending on the actual materials used, the history of past service and the proposed future service.

6.2.2 STANDARDS

(a) Appropriate grades of steel also should be used for the fabrication of any parts of the structure.

(b) All steels should be made by the open-hearth, electric furnace or basic oxygen process. Steels made by the air and mixed air-oxygen bottom blown basic converter processes, eg Bessemer or Thomas processes, should not be used. Steel plates, bars and sections and tubular shapes, etc. for primary structural members should comply with the requirements of the Canadian Standards Association Codes G40.20-M1978 and G40.21-M1978. The required inspection and testing in accordance with specification should be carried out by a competent inspection organization.

(c) In selecting the material to be used consideration should be given to the question of weldability. Material thickness, carbon equivalent, chemical composition, welding process, consumables and pre and post weld heat treatment, are interlinked variables and care should be taken to select a combination which ensures that cracking does not occur in the heat affected zone. Guidance on this subject is contained in the Canadian Standards Association Code W59-1977 Welded Steel Construction (Metal-Arc Welding).

6.2.3 NOTCH TOUGHNESS

Materials incorporated in any portion of the installation designated participating in the overall integrity and safety of the installation should have an appropriate standard of notch toughness. It is convenient to use the Charpy V notch test for determining the notch toughness and to select values of energy absorbed at a given temperature, on the basis of either a correlation with the results from a suitable type test which simulates the performance of the structure, or comparable operating experiences. As well, impact tests are not required of steels or weld zones (including the weld material) that are not part of the primary structure, providing such welds do not connect with the primary structure.

6.2.4 PLATE QUALITY

(a) Laminations: some steel plates contain planes of discontinuity or of weakness parallel to the plate surfaces. These planes of discontinuity, also called laminations, may be large or small depending in part upon the manufacturing process. Such laminations reduce the transverse strength of the plate as a whole and thus affect the integrity of the structure.

(b) Through thickness properties: welded connections transmitting loads to the surface of plate members can result in lamellar tearing of the laminated area of the plate, reducing the strength of the connection.

Where appropriate, the area of lamination and minimum through thickness strength values should be included in the quality specification of steel. A minimum level of ductility as measured by a reduction in the area of not less than 20 percent for a tensile test specimen should ensure freedom from lamellar tearing in most joint configurations although complex or highly restrained joints may require a higher value.

6.2.5 PIPE

All structural steel pipe should be seamless or manufactured with a relevant welding procedure and should not be expanded to more than 0.5 percent circumferential strain. Spirally welded pipe should not be used for any important load-carrying member. Reject or limited service process or line pipe should not be used in any part of the structure except for non-critical applications.

6.3 WELDING CONSUMABLES

6.3.1 GENERAL

The structure shall be fabricated using approved welding consumables and processes, eg appropriate grades of coated electrodes for manual welding; bare filler wires and fluxes for submerged arc welding; bare, solid or flux-cored, or flux-coated wire for gas shielded arc welding and cored wire for self-shielded arc welding.

6.3.2 REQUIREMENTS

All welding consumables should comply with the requirements of the certifying authority or with the relevant CSA standards. In general consumables of the low hydrogen potential type should be used for welding higher strength steels, with yield strengths greater than or equal to 300 MPa.

As well, any repair welding should utilize low hydrogen consumables.

6.3.3 EXTRA NOTCH TOUGHNESS

In certain areas where high levels of local plastic strain may occur (e.g. at complex joints) and where there are thick sections of simple welds, a higher level of notch toughness may be necessary. Under these circumstances it is recommended that welding consumables be selected on the basis of defect tolerance estimated from fracture mechanics tests (e.g. crack opening displacement and Well's wide plate tests prepared and tested according to the proposed production procedures) in conjunction with Charpy V notch impact tests. Minimum service temperatures should be accounted for, especially in those above water parts where temperatures may be considerably less.

6.3.4 EXISTING MOBILE INSTALLATIONS - INVESTIGATION OF RISK OF BRITTLE FRACTURE

In the case of an existing unit, if there are insufficient records available to assess the materials used and the methods of construction employed in relation to the standards given in the Guidance for Sections 6 and 7, an investigation will need to be made of the risk of brittle fracture. Resistance to brittle fracture is dependent upon the notch toughness of the material, the plate thickness of the material, the extent of cracklike defects present and the degree of local embrittlement at the tip of pre-existent defects (in the case of carbon and carbon-manganese steels, post-weld heat treatment in the stress relieving range of temperatures is effective in removing severe embrittlement arising as a result of welding and/or flame cutting operations).

Factors relevant to the investigation which should be considered are:

(i) the environmental criteria, in particular minimum temperature and extreme waves;

(ii) the impact loads to which the installation may be subject in use;

(iii) such information as is available on the materials used and the construction processes that were employed during fabrication of the structure;

(iv) evidence of repair or modification by welding or flame-cutting, especially where there is no information on subsequent stress-relief; and any history of fatigue or other cracks during service and of failure in primary or secondary structure.

An examination of the installation will be required and should be particularly directed to the detection of cracks in susceptible areas such as joints and load carrying structural details. Impact specimens should be obtained from samples of welded material and tested in order to ascertain the full transition temperature range of energy absorbed and fracture appearance. The possible existence of sub-standard conditions may be identified, bearing in mind that impact properties do vary over plates and welds, and that standard conditions may well become substandard if the material is thick or if high local plastic strain is applied. Where sub-standard conditions are found, more rigourous examination should be made to locate defects likely to be critical for crack initiation.

6.4 STRUCTURAL CONCRETE

6.4.1 GENERAL

Except where supplemented or amended by the provisions of this section the constituent materials for concrete, reinforcement and pre-stressing tendons should comply with the requirements of the Canadian Standard Association Codes. Large offshore structures are often built on remote sites which are long distances away from the normal sources of material supply. They also require large amounts of materials to be delivered in relatively short periods of time. In these conditions construction planning should pay close attention to the supply of materials to ensure the necessary consistency in their properties. For more northern areas such as the coast of Labrador particular attention should be paid to the frost resistances of the concrete used.

6.4.2 CEMENT

The cement used should be one of those specified in the relevant Canadian Standards Association Codes. Cements other than these should be used only where there is a specific reason for doing so and after a full consideration of their special characteristics and particular conditions of use.

In Portland cement a high tricalcium aluminate content should not be used unless it can be demonstrated that it does not have a deleterious effect on the quality of the concrete. In exceptional cases only, and where no practicable alternative exists, high alumina cement may be used, subject to the prior approval of the Certifying Authority.

6.4.3 AGGREGATES

General. Aggregates should comply with the requirements of the Canadian Standards Association Code CAN3-A23.1-M77 except where modified below.

Aggregates need to be chosen with more care than those for structures on land. To achieve the high strength required, the effect of the aggregate on the concrete's strength, density, drying, shrinkage, creep, thermal and other properties should be investigated to ensure that these properties are within the limits assumed in design.

Marine aggregates. The content of the chloride salt in the aggregate, expressed as equivalent anhydrous calcium chloride, should not exceed 0.1 percent by weight of the cement to be used in the concrete.

6.4.4 WATER

Sea water should not be used as a constituent material of concrete. In the absence of other adequate evidence of suitability, water should be tested in accordance with appropriate tests for water for making concrete.

6.4.5 ADMIXTURES

6.4.5.1 General

Air-entraining agents, workability aids and retarding agents may be used provided that suitable precautions are taken and it can be shown by tests that the product to be added will produce the required effect without in any way changing the other qualities required in the concrete or damaging the steel.

6.4.5.2 Dosage and Admixtures in Combination

Care should be taken to ensure that only the correct dosage is used and that the admixture is thoroughly dispersed throughout the concrete. The effects of over or under dosage should be made known to all those responsible for the approval, production and control of the concrete. If two or more admixtures are though. to be required in any one mix, the manufacturers of each should be consulted to comment about the expected effect on the concrete and reinforcement throughout its life. Trial mixes should be prepared and tests carried out to determine their acceptability. When the acceptability of the admixture is being assessed account should be taken of the likely consequences of over or under dosage.

6.4.5.3 Air-Entraining Agents

The use of air entraining agents in the concrete may be considered for the splash zone and for any other part of the structure which might be subjected to freezing and thawing when in a saturated condition.

Obtaining and controlling the required amounts of air is often difficult in concretes where the cement content exceeds 400 kg/m^3. Where this is so, long-term durability may be obtained with more confidence by the use of a uniform well-compacted concrete.

6.4.6 GROUT FOR PRESTRESSING TENDON DUCTS

Materials for grout should comply with the recommendation given in the Canadian Standards Association Code CAN3-A23.1-M77.

6.4.7 REINFORCEMENT

All reinforcing bars and prestressing wires and strands should be in accordance with the relevant CSA standards.

6.5 ALUMINUM ALLOYS

Aluminum alloys should comply with a recognized specification or with a recognized standard for use in a marine environment. For welded structural members, aluminum alloys should be used in the annealed condition and should be selected from those having a specified minimum 0.2 percent proof strength in the annealed condition not exceeding 130 N/mm^2. Higher strength alloys may be considered for non-welded construction. It should be noted that in the event of fire, aluminum loses strength and stiffness very rapidly.

6.6 OTHER MATERIALS

Other structural materials used in the construction of installations should have allowable stresses and load-carrying capacities determined in accordance with a standard recognized as appropriate to those materials.

References

1. National Association of Corrosion Engineers. Control of corrosion on steel fixed offshore platforms associated with petroleum production. RP-01-76.

SECTION 7 CONSTRUCTION

SUMMARY

This section deals with the methods of construction and association inspection and testing procedures that apply to both initial construction and subsequent modifications or repairs. It deals also with welding and the making and placing of concrete.

7.1 CONFORMITY TO DESIGN

An installation should be fabricated or constructed in accordance with drawings, specifications and other documents that have been approved by the certifying authority and these should be sufficient in number and detail to enable the installation to be built in accordance with the design, and in the case of an existing installation to enable the constructional details to be properly checked. All amendments made during fabrication or construction should be properly recorded.

7.2 CONSTRUCTION STANDARDS - STEEL

7.2.1 STANDARDS

The design should specify the standards or codes to be followed during construction. For mobile units these should include ship classification rules. For fixed units these should include requirements needed to meet the more onerous conditions applying to offshore installations in Newfoundland and Labrador waters. Reference may also be made to CSA Standard W59-1977 and any other code recognized for certification purposes (references 1 and 2).

7.2.2 INSPECTION AND TESTING

7.2.2.1 Scope of Procedures

The testing procedures should be laid down in the design drawings and other documents. When additional tests are required, a procedure should be agreed between the parties

concerned. Inspection and testing should be conducted throughout all stages of construction. Such inspection and testing procedures should clearly delineate the extent of the test and the methods to be used and should include soundness and proper placement of materials, quality control of welds, application of coatings, tolerances, placing of foundation piles and control of concreting, etc.

7.2.2.2 Methods of Inspecting Steel Structures and Components

Radiographic procedures and film interpretation should be made by competent persons in accordance with recognized standards. Magnetic particle inspection of welds should be carried out in accordance with the requirements in references 3-5, ultrasonic inspection should be performed in accordance with appropriate standards; personnel carrying out tests and interpreting the results should be qualified and experienced in the appropriate type of inspection (references 6-11).

7.2.2.3 Testing Arrangements for Steel

It is permissible to defer the soundness testing until structural elements are ready for assembly, when the area meeting the required soundness level may be selected by shifting or rotating unsuitable areas away from the stressed zones. Normally all welded joints critical to the integrity of the structure should be subjected to 100 percent non-destructive testing during construction, but double-sided longitudinal butt welds in tubulars made by automatic welding processes may be subject to 50 percent non-destructive testing except in the case of cans for nodes. The scope of tests of other welds should be agreed with the certifying authority. All weld testing should be fully prescribed by the drawings and/or the specification applying to the construction (see also 7.2.4).

7.2.2.4 Testing and Inspection of Concrete Structures and Components

Testing and inspection of concrete structures and components should be carried out in accordance with CAN3-A23.2-M77. Records should be maintained for all routine tests on concrete, and also, where appropriate, of the full prestressing programme.

7.2.3 MODIFICATIONS AND REPAIRS

All modifications and repairs should be subject to the requirements specified for initial fabrication or construction

in documentation, conformity to design, construction standards, supervision, examination and testing (7.1-7.2.2 above). Materials and procedures used should be compatible with those employed in the initial fabrication or construction.

7.2.4 OPERATIONS MANUAL - PRECAUTIONS

The Operations Manual should advise on any special precautions or considerations to be employed when repairs or alterations to the structure are to be carried out at sea. It should include information on the particular subject, guidance when action may be needed and instructions for carrying out the remedial work.

7.2.5 WELDING

Welding procedures should be verified through sample weld preparation and testing. The purpose of this testing is to determine positively whether the materials and methods prescribed in a procedure specification will, in fact, produce welded joints that meet the application and specification requirements. Welding procedure should be tested in four steps:

(a) Preparation and welding of suitable samples. In addition to any other samples required a procedure should be specifically devised to represent the welding conditions at Y and K type joints. It is considered that the standard all-positional weld procedure does not adequately reflect the difficulties of cross-over type welds.

(b) Testing of representative specimens. The range of plate thickness qualified by a welding procedure is to be $t \pm 0.5t$ where t is the thickness of the procedure test plate.

(c) Evaluation of overall preparation, welding, testing and results (see 7.2.5.2 below).

(d) Approval.

Complete approval for the introduction of a production welding procedure combines the above four steps with three others:

(e) Testing and approval of the welders to determine their ability to deposit sound welds (see 7.2.5.1 below).

(f) Providing suitable supervision.

(g) Providing suitable inspection during and following completion of welding.

Limitations imposed by the essential variables of the approved procedure must be adhered to in production welding. Weld procedures for steels with specified notch toughness properties should require testing of the welds to ensure that the weld joints meet the notch toughness requirement.

7.2.5.1 Welder Qualifications

The procedure for testing welders may be patterened after CSA W117.2-1974 and W47.1-1973, with the additional requirement of mechanical testing if necessary (reference 7). In general, these tests should be conducted using specimens similar to those required for welding procedure approval and should encompass a level of welding difficulty at least equal to that to be encountered in all production welding. Testing of specimens should be conducted by qualified testing personnel.

7.2.5.2 Production Welding

Unless otherwise specified, welding should comply with the requirements of CSA W-59-1977 (reference 7). As a general rule all welds in critical regions should be a full penetration double-sided type. Welds made from one side only, without backing, may be permitted provided that welder and welding procedure qualifications encompass such welds.

7.2.5.3 Stress Relief

The post-weld heat treatment (PWHT) of a weldment can reduce the risk of brittle fracture during service, and may increase the fatigue life. The advantages of such heat treatment increase with increase in section thickness and complexity of the construction. For relatively simple constructions in normalized carbon or carbon-manganese steels the following requirements are regarded as limiting and are not to be exceeded unless there is adequate and reliable supporting evidence from service or the results of relevant laboratory tests:

(a) Tubulars for nodes, internal ring supports between cans and braces, are to be PWHT on completion of the welding of the longitudinal seam when the actual thickness exceeds 40mm for parts of the structure exposed to the atmosphere and 50mm for submerged parts.

(b) Where the actual thickness of a brace welded to a can exceeds 40mm and is exposed to the atmosphere in service, or 50mm for submerged nodes, the node is to be PWHT on completion of all welding.

(c) Butt welds between cans and braces or in the length of a brace are to be PWHT when the thickness exceeds the values quoted above. In these cases the reference thickness is that of the greater at the position of the butt weld. However, where a stub has a thickness in excess of the limiting value, and the associated brace is below that thickness, the end of the stub may be tapered to the thickness of the brace.

Note: These requirements are for structures whose minimum service temperatures are not expected to be lower than minus 10 degrees celcius and when this minimum service temperature is exceeded, reliable supporting evidence from service or the results of relevant laboratory tests are necessary.

7.2.5.4 Construction Test Procedures

Consideration should be given to the use of procedures which relate the average Charpy V notch energy of a weld or heat affected zone to the guaranteed minimum yield stress of the weaker of the steels being welded and the thickness of the thickest test plate.

7.2.5.5 Welding Supervision and Inspection

During construction, care should be taken to ensure that the properties of the consumables are maintained and that production welds are made strictly in accordance with the approval procedures.

7.3 CONSTRUCTION STANDARDS - CONCRETE

Except where supplemented by the provisions of this section, concrete construction should be carried out in accordance with the recommendations of CSA CAN3-23.3-M77 code for the Design of Concrete Structures for Buildings and CSA CAN3-23.1-M77, Concrete Materials and Methods of Concrete Construction.

7.3.1 CLASSIFICATION OF CONCRETE MIX

Normally the concrete should be a designed mix. Prescribed mixes may be used only where it can be shown that the properties of the finished concrete are in every way adequate to achieve the intended purpose.

7.3.2 CONCRETE STRENGTH

The concrete strength shall be as specified in Part II Section 4.2.3.2.

References

1. American Petroleum Institute. Recommended practice for planning, designing and constructing fixed offshore platforms. API RP 2A, 1981.
2. American Institute of Steel Construction. Manual of steel construction.
3. British Standards Institution. Methods of magnetic particle testing of welds. BS 4397.
4. American Society of Testing and Materials. Dry power magnetic particle inspection. ASTM Specification E 109-63.
5. American Society for Testing and Materials. Wet magnetic particle inspection. ASTM Specification E 138-63.
6. American Society for Non-Destructive Testing. ASNT Standard TC-1A.
7. American Welding Society. Structural welding code - AWS D1.1.
8. British Standards Institution. Calibration blocks and recommendations for their use in ultrasonic flaw detection. BS 2704.
9. British Standards Institution. Methods of ultrasonic examination of welds. BS 3923.
10. British Standards Institution. Methods of assessing the performance characteristics of ultrasonic flaw detection equipment. BS 4331.
11. British Standards Institution. Methods of non-destructive testing of plate material. BS 4336.

SECTION 8 EQUIPMENT

SUMMARY

This section applies to all fixed equipment installed or erected on an offshore installation, whether initially or at a subsequent date. Equipment is to be considered in respect of its relationship to the safety of the structure and the safety and health of those on board. General requirements are given relating to all such equipment, and specific functional requirements for ballasting, bilge pumping, dynamic positioning, mooring, elevating, ventilation, heating and cooling, lighting and emergency power supply. The requirements of Section 8 are supplemented by Part III and Part IV of this publication.

8.1 GENERAL REQUIREMENTS

8.1.1 STANDARDS

All equipment, whether installed initially or at a subsequent date should be manufactured to a relevant standard, code or specification and written confirmation of this together with appropriate test certificates should be obtained from the manufacturer. Detailed requirements for electrical equipment and for mechanical equipment are contained in Parts III and IV respectively.

8.1.2 SUITABILITY AND SAFETY IN OPERATION

All equipment should be suitable for its intended purpose. Where relevant, equipment should be designed having regard to its intended use with or near other equipment and for its safe use under all known operating conditions, including overload if anticipated. Where relevant, equipment should have efficient control systems, guards, fences and shields.

8.1.3 LOCATION OF EQUIPMENT

Equipment should be located with safety in view: any equipment located in hazardous areas should be suitably protected for installation in such areas.

8.1.4 ACCESS TO EQUIPMENT

People should have safe means of access to and egress from places where they will live and work. If these places are potentially dangerous, suitable protection or working cabins should be provided.

8.1.5 INSTALLING OF EQUIPMENT

8.1.5.1 Individual Items

It is essential that each item of equipment should be installed properly. Particular consideration should be given to the effectiveness of mountings for cranes, fixed lifting appliances and derricks (including any drilling derrick); to the safe fixing of air intakes and exhausts of engines and compressors; to the safe placing of equipment in relation to the electrical system; to the segregation of piping systems, particularly pressure systems; to the inclusion of master controls for stopping machinery, for shutting off fuel supplies and for closing combustion spaces.

Consideration should also be given to the safe installation of electrical apparatus and conductors including the supply system, plant, machines and fittings.

8.1.5.2 Items Taken Together

The installation and disposition of each item of equipment should also be considered in relation to other items so as to reduce to a minimum any potential danger to the installation and the people thereon. The whole of the equipment should be safely designed, constructed and installed.

8.2 BALLASTING

Where necessary to permit operations to be carried on safely, a mobile installation should be provided with a ballasting system which is capable of efficiently trimming and adjusting the draft of the unit. Ballast tanks and associated equipment should be in accordance with the relevant provisions of ship classification rules, with any additional capacity needed for operations on station.

It should be possible to efficiently pump water at all angles of trim and heel that the unit is designed for. At least one pump in each pump room/pontoon should be capable of

operating off auxiliary (emergency) power. There should be adequate measures to ensure that the emergency generator cannot be overloaded in this configuration. There should be indication on the control panel that pumps are running and that water is being transferred. There should be the capability to start the pumps locally if the control panel is shut down. There should be the capability to manually operate the sea suction valves in the event of a complete power failure. There should be means to disconnect the control functions of the control panel preferably leaving the indicator functions intact.

All valves should be labelled such that an untrained person would operate them if necessary under instruction from the control room. There should be a copy of the mimic diagram on impervious material in the pump room. A set of equipment needed to manually operate the values should be kept in every compartment in which there are values. The walkway gratings should be fitted according to standard good ship practice for engine room gratings with convienient access to valves.

The operation manual shculd contain details of automatic and manual operation of the system.

8.3 BILGE PUMPING

All units when afloat should have adequate bilge pumping systems and means of draining all compartments and watertight sections. Those compartments used for containing liquids should have their own separate pumping and drainage system. The capacity and arrangement of the system should follow the relevant provisions of ship classification rules. The system should be operable at all angles of trim and heel that the unit is designed for.

8.4 DYNAMIC POSITIONING

Any dynamic position system should be shown to be adequate for maintaining the installation in position and on correct heading in all design operating conditions. Its control equipment should be fitted with a device able to record loss of position and give a visual or audible warning if loss of position becomes critical. Manual over-ride should be provided.

8.4.1 POWER SUPPLY TO DYNAMIC POSITIONING UNITS

The power supply to the dynamic positioning units should be divided so that in the event of failure of a single generating unit, there will still be sufficient power to maintain position.

8.5 MOORING

8.5.1 CAPABILITY OF SYSTEM

In considering any mooring system relevant forces should be assessed, calculations made and, if possible, model tests undertaken. As well, all global and local dynamic effects shall be taken into account. The system should be adequate to hold the installation on location in the most severe environmental conditions in which operations can be carried out. Extreme stresses and restoring forces should be taken into account in the means by which cables are attached to the structure.

8.5.2 DETAILS OF SYSTEM

8.5.2.1 Selection of Anchors and Cables

When selecting the strength and type of anchor required due regard should be paid to the mooring pattern to be employed, the expected nature of the sea bottom and the environmental conditions likely to be experienced.

8.5.2.2 Materials for Ropes and Cables

Wire ropes for tow and mooring lines should be of flexible construction and conform with recognized marine practice. The material used for chain cables and the form and proportion of the links and shackles should be adequate for the service for which they are intended; marine practice should be followed.

8.5.2.3 Testing and Marking of Anchors, Cables and Ropes

Anchors, chain cables and wire rope cables should be tested and marked as is required for such equipment on Canadian registered ships.

8.5.2.4 Windlasses and Winches

For chain cables, windlasses of sufficient power suitable for the size of the chain cable should be provided. Winches for wire ropes should have adequate paying-on gear and sufficiently large drums to accommodate all the wire (but see 8.5.2.6). All winches and chain windlasses should have adequate power and brakes to maintain control of the cables at all times. They

should be securely fitted to strong points capable of withstanding a load at least equal to the strength of the cable.

8.5.2.5 Hawse Pipes and Chain Lockers

Hawse pipes where fitted should not allow the entry of water into the unit and should be of adequate thickness and size and securely fastened; extra wearing strips should be provided where a unit remains at anchor for long periods. Chain lockers should be of a capacity and depth to provide a direct lead into the chain pipes when the cable is stowed. A separate locker should be provided for each chain with a suitable means of attachment of the chain.

8.5.2.6 Items Not Held on Board The Installation

If appropriate, parts of the mooring system (eg extension wires or chains for use in deeper waters) may be carried on board supporting vessels which are used to assist in laying and attending to moorings when the installation is on location. Such items should be considered as part of the whole system when assessing its adequacy.

8.6 VENTILATION, HEATING AND COOLING

8.6.1 SCOPE OF SYSTEMS

Ventilation, heating and cooling should be provided for all areas except unmanned areas, such as ballast tanks where flammable vapours are not likely to occur. To ensure that inlet louvres are not obstructed by snow, downward-looking cowls should also be provided at all inlets. To balance pressure, similar cowls should also be provided to outlet louvres. There should be separate ventilation for any area where flammable or noxious gases may occur, with inlet and exhaust openings so segregated that cross-contamination of the ventilation of other areas is prevented. Where appropriate, there should be purging air systems for removal of gases from spaces within hazardous areas and pressurization of working spaces, living spaces and accommodation spaces to prevent the ingress of escaping gases.

8.6.2 DETAILED ASPECTS

The following aspects should be considered:

(a) the maintaining of suitable temperatures, supplies of fresh air, humidity conditions and of acceptable levels of noise;

(b) thermal insulation and double glazing where necessary;

(c) air filtration;

(d) the provision of standby equipment and of master controls for stopping ventilation in an emergency;

(e) adequate discharge of exhaust air under all possible wind conditions;

(f) separate air extraction from kitchen, laundry and sanitary areas;

(g) maintaining a suitable positive air pressure in living, accommodation and enclosed working areas, where practicable;

(h) detection of escaped gas from hazardous areas and at fresh air inlets to safe areas;

(i) continuous monitoring of gas levels in enclosed hazardous areas;

(j) alarm systems to warn when there are dangerous levels of gas accumulation in enclosed hazardous areas;

(k) adequate ventilation of enclosed hazardous areas to be capable of a complete air change in two minutes

(l) maintaining a suitable negative air pressure in the enclosed hazardous areas relative to the surrounding areas;

(m) keeping any room used to monitor mud properties or to examine drill cuttings pressurized and fitted with air alarm to warn the occupants when there is a loss of pressure;

(n) where a machinery room is equipped with a mechanical ventilation system it should have two waster controls, one located outside the machinery room.

8.7 LIGHTING

Lighting systems should provide a sufficient level of illumination in all areas likely to be manned and where apparatus is installed. Working areas, corridors, stairways, ladders, walkways and landing should be especially well lit. For lighting of the helicopter deck see Section 5.3. The systems should be so arranged that escape routes, embarkation areas and any control panel or operational station which would be required to be manned in the event of loss of normal electrical power should be capable of being supplied from the emergency power source.

8.8 EMERGENCY POWER SUPPLY

8.8.1 SERVICES TO BE SUPPLIED

There should be equipment capable of supplying emergency electrical power and consisting of at least:

(a) prime mover/generator to supply the following services for a period of at least 24 hours:

emergency lighting;
the helicopter landing area perimeter;
radio installation;
warning systems;
fire extinction systems;
alarm and public address systems;
diving equipment dependent on an electrical supply;
navigational lighting;
navigational aids;
pumps and valves that are essential for maintaining the installations trim;
well disconnect systems that are dependent on electrical power;

(b) battery of accumulators to supply the following services for a period of at least 24 hours after all other power sources have failed:

aircraft warning lights;
navigational lights;
obstruction lights;
communications and navigation control room lighting;
marine radio

8.8.2 EQUIPMENT TO BE PROVIDED

To meet the requirements in 8.8.1 there should be individual emergency power supply arrangements for the various services or a general source of emergency power supply connected to the services.

8.8.3 STANDARDS FOR EMERGENCY POWER SOURCES

The emergency source of electric power should consist of a battery of accumulators, and a generator powered by a compression ignition engine or gas turbine operating on a fuel with a flash point greater than 43°C. The emergency equipment and any associated fuel storage should be situated outside any hazardous area, and be independent of, and remote from, the main power supply. It should be contained in a room with an exit to the open air. It should come into operation automatically if the main power supply fails, but, if the emergency source of power is a generator, the arrangement should be such that the set can be started independently of the automatic mechanism. It should be capable of running without supervision, i.e. there should be an automatic means of lubrication and an adequate supply of fuel oil where necessary.

8.9 GENERAL ALARM SYSTEM

All installations should be equipped with an alarm system designed to show personnel at all times the general state of the unit as a whole.

8.10 OPERATIONS MANUAL: EQUIPMENT

The Operations Manual should contain brief particulars of all significant equipment on the installation, the major general arrangements including a general arrangement drawing of the equipment on the platform, flow sheets and instruction for the installation, operation and maintenance of the equipment and the platform. It should also include the design parameters including pipework (see Part IV).

For equipment in paragraphs 8.2-8.5 there should be included additionally, with respect to the range of operating conditions expected, guidance when action may be necessary and instruction on the action to be taken in the installation's various modes of use whether on station, placing on or removal from station, or movement between stations.

8.11 FIRE FIGHTING SYSTEMS

8.11.1 MAIN FIRE FIGHTING SYSTEM

All installations should be equipped with

(a) a main fire fighting system that includes:

(i) at least two fire pumps each independently driven and each having sufficient capacity to produce two continuous streams of water, at a minimum pressure of 275 kilopascals and at a minimum rate of 200 litres per minute, simultaneously from any two hydrants,

(ii) hydrants sufficient in number and located so that all living, working, machinery and storage areas of the installation are within 15 metres of a hydrant and within 45 metres of a second hydrant,

(iii) a hydrant located on a side of the compartment containing the main internal combustion engines on the installation and a second hydrant located on that side of the compartment opposite the site of the first hydrant, each equipped with hoses and fog nozzles or applicators suitable for spraying water on oil,

(iv) adequate protection against freezing for all components of the system;

(b) where fire pumps are located above the water line of the installation, a water storage tank installed at the main deck level of the installation that is

(i) connected to at least two of the pumps referred to in Section 8.11.1 (a) (i);

(ii) of sufficient capacity to supply water to two outlets simulataneously at the rate specified in section 8.11.1 (a) (i) for a minimum period of fifteen minutes without replenishment;

(iii) equipped with a means of replenishment that is reliable and that has a flow capacity greater than the combined capacity of the water pumps connected to it, and

(iv) equipped with an alarm to warn that water in the tank is not sufficient to comply with Section 8.11 (b) (ii).

8.11.2 TOTAL FLOODING SYSTEM

Every engine room, boiler room or paint storage room on all installations should be equipped with a carbon dioxide total flooding system or a total flooding system that gives an equivalent fire protection. Such system should:

(a) include devives to warn personnel before activation;

(b) be controlled by a device located outside the room.

All rooms referred to in this section should be designed so that all ventilators, ports or other openings can be closed.

8.12 ICEBERG DETECTION

Each installation should have both X band and S band radar systems as part of its iceberg management system. Each radar system should be comprised of an antenna, a transmitter/receiver unit and a display unit and any necessary ancilliary equipment.

There should also be a backup X band and S band radar system comprised of a transmitter/receiver unit and a display unit and any necessary ancilliary equipment. The back-up system should be fully operational in a standby mode.

PART III REQUIREMENTS FOR ELECTRICAL APPARATUS AND CONDUCTORS

(Part I and Part II, Section 1, also apply)

1. Electrical installations should comply with IEEE Recommended Practice for Electrical Installations on Shipboard Standard 45-1971 or with such other code or standard that is equivalent or superior thereto.

2. The Institution of Electrical Engineers REGULATIONS FOR THE ELECTRICAL AND ELECTRONIC EQUIPMENT OF SHIPS (fifth edition, 1972) may soon be modified to include offshore installations.

3. The British Standards Institution has already BS 5345 Part I, SELECTION, INSTALLATION AND MAINTENANCE OF ELECTRICAL APPARATUS FOR USE IN POTENTIALLY EXPLOSIVE ATMOSPHERES.

4. Further, the International Electro-technical Commission (IEC), Technical Committees TC31 and TC18, have published documents which deal with relevant technology.

5. The requirements of the Offshore Installations (Construction and Survey) Regulations 1981 will be met if the recommendations by IEEE, BSI and IEC are compiled with, where relevant.

6. In the absence of relevant formal publications from the three societies (IEEE, BSI, IEC) existing good practice for electrical engineering should be used.

7. All electrical equipment used on a installation should be designed and located in such a manner as to avoid the creation of any hazard.

8. All electrical equipment on an installation should be explosion proof or pressurized where it is installed

   (a) in any open space within 15 metres horizontally of the rotary table of the drilling unit;

   (b) less than 3 metres above the drill floor of the drilling unit or less than 10 metres below the drill floor, or

   (c) within 3 metres of a mud ditch, shale shaker, degasser or mud tank.

9. The primary circuits from the power plant serving on installation should be equipped with at least two manual shutoff switches each at a different location.

Relevant Documentation

(a) The Institute of Electrical and Electronics Engineers (IEEE) Recommended Practice for Electrical Installations on Shipboard Standard 45-1971.

(b) Institution of Electrical Engineers Regulations for the Electrical and Electronic Equipment of Ships, 5th. edition 1972 (under revision).

(c) BS 5345. Selection, installation and maintenance of electrical apparatus for use in potentially explosive atmospheres. Part 1: Basic requirements for all parts of the code.

(d) IEC publications: 79.0
79.1
79.2
79.3
79.4
79.5
79.6
79.7
79.8
79.9
79.10

PART IV REQUIREMENTS FOR MECHANICAL EQUIPMENT

(Part I and Part II, Section 1, also apply)

1. FIRED AND UNFIRED PRESSURE VESSELS

1.1 The design and construction of pressure vessels shall be in accordance with Canadian Standards Association Code B51-1975 for the Construction and Inspection of Boilers and Pressure Vessels.

Every fired and unfired pressure vessel should have:

(a) a clearly visible distinguishing mark;

(b) its maximum allowable pressure marked so as to be clearly visible;

(c) means for fitting a test pressure gauge;

(d) a handhole or manhole to allow the interior to be thoroughly cleaned and examined;

(e) a gauge to clearly indicate the level of the contents for unfired vessels and two separate means for fired vessels with liquid contents.

(f) a valid certificate issued by a competent authority to show that it has been examined externally and internally and tested as appropriate at intervals of time to be determined by the certifying authority.

Note: Alternative methods are permissible for (d) and (e) above.

1.2 Where a pressure vessel is fitted with safety valves or bursting discs to prevent the vessel from being worked at a pressure greater than the permissible working pressure, the following should also be fitted:

(a) a suitable stop valve on the supply line;

(b) a pressure gauge to clearly indicate the maximum working pressure.

1.3 Safety valves should be designed and constructed to be suitable in every way for the duty assigned to them.

1.4 If the design of the safety valve is such that liquid can collect on the discharge side of the valve discs the safety valve should be fitted with a drain at the lowest point where liquid can collect.

1.5 Where bursting discs as a pressure relieving device are fitted, they should be of a suitable type and clearly marked so as to identify the disc with its duty.

1.6 If on exposure to a naked flame the contents of a pressure vessel could cause an explosion, the flow of gases resulting from the lifting of the safety valve should be vented to a safe place.

1.7 All pressure vessels should be constructed to withstand the maximum pressure available, and safety relief valves or equivalent installed to provide a suitable means of reducing the pressure to a safe working level.

1.8 All pressure vessels should be fitted with suitable manual or automatic means to drain the vessel.

1.9 The vent line from any safety valve should be self-draining.

1.10 Where it is considered necessary to fit stop valves in conjunction with pressure relieving device, there should be two independent pressure relieving devices fitted, each capable of preventing overpressure. Stop valves should be of the full flow type capable of being locked in the full open position and so arranged that when one pressure relief device is isolated the other will be open to the pressure vessel system.

2. PIPING SYSTEMS

2.1 All piping runs should be clearly identified in accordance with CSA B53-1958 code for Identification of Piping Systems or other acceptable means. A key to the coding should be included in the Operation Manual.

2.2 All pipework should be constructed to withstand the maximum pressure available.

2.3 Pipework material should be suitable for its intended use.

3. LIFTING APPLIANCES

3.1 All lifting appliances should have their safe working load clearly marked in a conspicuous position.

3.2 All cranes should have fitted:

(a) a device to prevent the hook block from coming into contact with the jib head;

(b) an automatic safe load indicator to give the crane operator a clear and continuous visual warning when the load being carried exceeds a figure not less than 90 percent and not more than 97 1/2 percent of the safe working load of the crane at that radius and to give the crane operator and any other person in the immediate vicinity of the crane a continuous sound warning when the load exceeds a figure not less than 102 1/2 percent and not more than 110 percent of the safe working load of the crane at that radius;

(c) cut-outs which automatically arrest the hoisting and luffing motions of the crane, by cutting off the power before or when the load being carried exceeds by 10 percent the safe working load of the crane at that radius, but which do not prevent the removal of the excessive load; or as an alternative:

an alarm which warns the crane operator and any other person in the immediate vicinity when the hoisting and luffing motions of the crane carrying the load exceed by 10 percent the safe working load of the crane at that radius;

(d) a device which automatically and continuously weighs the load being carried to an accuracy of ± 5 percent and registers the weight on an indicator clearly visible to the crane operator;

(e) an efficient brake or brakes capable of preventing the fall of a suspended load and of effectively controlling the speed of hoisting or lowering the load.

3.2.1 Cranes that have been overloaded: are cranes which have been exposed to a load moment exceeding 110 percent of the

permitted load moment, and shall be thoroughly examined by a competent person before it is again taken into use.

3.3 All crane controls should be clearly marked to show their respective functions and so arranged that the crane cannot be operated, or a suspended load allowed to fall, by accident.

4. COMBUSTION EQUIPMENT AND COMBUSTION ENGINES

4.1 Generally, combustion equipment and combustion engines should not be located in areas designated as hazardous. Where it is necessary to use a space in a hazardous area to house combustion equipment or engines, sufficient precautions must be taken against the risk of ignition. If pressurization of this space is done to make it safe, the following criteria will apply as a minimum:

(a) pressurization air is to be taken from a safe area;

(b) a pressure of 70 N per square metre is to be maintained with one door giving access to the space fully open; alternative equivalent standards such as NFPA 496 are also acceptable;

(c) an alarm system should be fitted to indicate pressurization failure and there should be a written procedure to show what course of action will be taken should a failure occur;

(d) an air-lock system is to be fitted.

4.2 Efficient spark-preventing equipment should be fitted to the exhausts from all combustion engines and equipment, except gas turbines. A means should be provided to give a warning when the supply of water fails to water-cooled spark-preventing equipment. All exhausts should be led to a safe area.

4.3 Air intakes for combustion engines and equipment should be taken from a safe area. An area for air intake is not safe when:

(a) it is located inside any area where combustible gases may accumulate;

(b) it is located less than 20 metres from the centre line of the well-bore;

(c) it is directed toward the well bore or any other potential source of combustible gases.

Air intakes of combustion engines, except gas turbines, situated in an enclosed space containing a flammable gas supply should have an automatic device fitted to prevent over-speeding in the event of accidental ingestion of flammable gas.

4.4 All enclosed machinery spaces containing combustion engines or equipment should be provided with sufficient mechanical ventilation to prevent danger to personnel due to escape of products of combustion, fuel gas or other harmful agents.

5. GENERAL

5.1 Efficient means, suitably located, should be provided for cutting off the power to any machinery.

5.2 Evidence should be available to show that in respect of electrical generating sets, vibration stresses in crank and motor shafts resulting from torsional critical speeds are not excessive.

INDEX OF STANDARDS AND CODES

In all cases Standards, Codes, and Manuals are to be the editions current at the time of use.

CANADIAN STANDARDS ASSOCIATION

CAN3-S16.1-M78	Steel Structures for Buildings - Limit States Design.
CAN3-S6-M78	Design of Highway Bridges.
CAN3-S6S1-1980	Supplement No. 1 to CAN3-S6-M78, Design of Highway Bridges.
CAN3-S6S2-1981	Supplement No. 2 to CAN3-S6-M78, Design of Highway Bridges.
CAN3-A23.1-M77	Concrete Materials and Methods of Concrete Construction.
CAN3-A23.2-M77	Methods of Test for Concrete.
CAN3-A23.3-M77	Code for the Design of Concrete Structures for Buildings.
CAN3-23.4-M78	Precast Concrete - Materials and Construction.
G40.21-M1978	Structural Quality Steels.
G40.20-M1978	General Requirements for Rolled or Welded Structural Quality Steel.
W59-1977	Welded Steel Construction (Metal-Arc Welding).
W48.1-M1980	Mild Steel Covered Arc-Welding Electrodes.
W48.2-M1980	Corrosion-Resisting Chromium and Chromium-Nickel Steel Covered Welding Electrodes.
W48.3-1976	Low-Alloy Steel Arc-Welding Electrodes.
W48.4-M1980	Solid Mild Steel Filler Metals for Gas Shielded Arc-Welding.
W48.5-1970	Mild Steel Electrodes for Flux Covered Arc-Welding.
W48.6-M1980	Bare Mild Steel Electrodes and Fluxes for Submerged Arc Welding.
CAN3-A5-M77	Portland Cements.
CAN3-A266.1-M78	Air-Entraining Admixtures for Concrete.
CAN3-A266.2-M78	Chemical Admixtures for Concrete.
CAN3-A266.3-M78	Pozzolanic Mineral Admixtures for Use in Portland Cement Concrete.
CAN3-A266.4-M78	Guidelines for the Use of Admixtures in Concrete.
A362-M1977	Blended Hydraulic Cement.
A363-M1977	Cementitious Hydraulic Slag.
G30.3-1972 (R 1979)	Cold-Drawn Steel Wire for Concrete Reinforcement.

INDEX OF STANDARDS AND CODES

In all cases Standards, Codes, and Manuals are to be the editions current at the time of use.

CANADIAN STANDARDS ASSOCIATION

G30.4-1976	Fabricated Deformed Steel Bar Mats for Concrete Reinforcement.
G30.5-1972 (R 1979)	Welded Steel Wire Fabric for Concrete Reinforcement.
G30.12-M1977	Billet-Steel Bars for Concrete Reinforcement.
G30.14-1972 (R 1979)	Deformed Steel Wire for Concrete Reinforcement.
G30.15-1972 (R 1979)	Welded Deformed Steel Wire Fabric for Concrete Reinforcement.
G30.16-M1977	Weldable Low Alloy Steel Deformed Bars for Concrete Reinforcement.
G30.17-1976	Welded Steel Plain Round Bar or Rod Mats for Concrete Reinforcement.
W47.1-1973	Certification of Companies for Fusion Welding of Steel Structures.
W117.2-1974	Code for Safety in Welding and Cutting (Requirements for Welding Operators).
B51-1975	Code for the Construction and Inspection of Boilers and Pressure Vessels.
B53-1958	Code for Identification of Piping Systems.

NATIONAL RESEARCH COUNCIL

NRCC No. 17303	National Building Code of Canada, 1980.
NRCC No. 17724	The Supplement of the National Building Code of Canada, 1980.

CANADIAN GEOTECHNICAL SOCIETY

Canadian Foundation Engineering Manual

OFFSHORE INSTALLATIONS: GUIDANCE ON DESIGN AND CONSTRUCTION

APPENDICES

FINAL DRAFT, AUGUST, 82

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APPENDIX A: ICE LOADS

APPENDIX A: ICE LOADS

A.1 GENERAL

There are zonal and seasonal variations in ice activity offshore Newfoundland and Labrador. Ice begins to form on the northern Labrador coast in early December. The extent of ice advances southward as winter progresses. Second and multi-year ice move into the region from the Arctic and rafting and ridging of first, second, and multi-year are present.

The presence of ice considerably influences offshore installations planning, design, and operations. The planning considerations should relate to the low temperatures and the presence of ice together with the other offshore environmental conditions prevalent in more temperate areas.

The prediction of ice loads on a structure is a complicated ice-structure interaction problem and depends on a number of parameters of the ice, the structure and the environment.

A.1.1 PLANNING CONSIDERATIONS

i) Offshore installation. The severity of ice conditions and it's management would influence the type of the installation. The installation should be capable of withstanding the ice; destroying, weakening or diverting the ice; or have procedures to move away from the ice. Proper surveillance of ice conditions would be necessary for operations involving management of ice.

ii) Arrangement of equipment. Due consideration should be given for the low temperatures, limited daylight and limited access to the structure and facilities. Storage space and equipment redundancy should be considered to account for limited accessibility in adverse weather. Insulation considerations should account for problems associated with the contact of heated spaces with perma-frost and ice. The location, orientation and support of buildings and equipment should be planned to avoid snowdrift and ice accumulation.

iii) Subsystems. The pipeline risers and conduits should be protected from interaction with ice by encasement and burial where ice scours are possible. Other methods may be considered if they can be demonstrated to provide effective protection from ice interaction. Adequate provision should be made for possible thermal expansion and contraction due to dissimilar temperatures between subsystems and temperature changes for various operating conditions (operating, shutdown, etc.).

iv) Transport and installation. For installations that are not fabricated at site the effect of change in temperature and the ice conditions during transit and installation should be considered.

v) Clearance. Due consideration should be given for clearance from ice movement and pile up/accumulation. Defence mechanisms should be planned where possibilities of ice ride up exist.

vi) Safety. Where enclosed work spaces are needed due to the adverse environment they should be planned with special provisions in the event of fire, explosions and gas accumulation.

A.1.2 CURRENT TECHNOLOGY

Much effort has and is being expended to explain the behaviour of ice interaction with structures and still forms a part of the current research effort. Care should be taken in interpreting available information. The mechanics of ice loading on offshore structures are not fully understood, and confirmation of theoretical models through full-scale measurement is almost entirely lacking. Some theories depend on very severe over-idealizations, and should be used cautiously.

A.2 ICE AND ICEBERG PROPERTIES

A.2.1 GENERAL

This section provides guidelines for the selection of ice properties assembled from publically available literature sources.

A.2.2 SEA ICE

A.2.2.1 PHYSICAL PROPERTIES

Sea ice is a crystalline material. Its properties will depend on the size and orientation of its crystals and its salinity, temperature, density and impurities (Assur, 1958; Schwarz and Weeks, 1977; Peyton, 1968 and Graff, 1977).

a) Crystal Structure. Sea ice, as it grows as a sheet on the ocean's surface, generally becomes a columnar-grained

anisotropic material; however, the presence of impurities (e.g., sediments) can promote "seeding" which forms small granular ice crystals. A preferred crystal orientation may develop in the horizontal plane of columnar-grained ice, probably caused by under-ice currents. Crystal sizes generally increase as the ice sheet grows thicker.

b) Salinity. Salt entrapped in sea ice is concentrated in small liquid brine pockets between ice crystals and is a function of the temperature. First-year ice salinities normally range from 3-6 parts per thousand (Weeks and Assur, 1967) but the salinity of multiyear ice is considerably lower, for as the ice ages the brine calls coalesce and brine drains from the ice. Ice of low salinity may be found near river deltas.

c) Temperature. The temperature of ice is a function of the air and sea temperatures and varies across the depth. Some laboratory evidence suggests that the uniaxial strength of sea ice is a function of its brine volume, v_b, which is a function of temperature. An empirical relationship for the brine volume, developed by Frankenstein and Garner (1967) is

$$v_b = S(0.532 - \frac{49.185}{T}) \qquad (1)$$

where S = salinity in parts per thousand (ppt);
T = ice temperature in ^{o}C; and
v_b = brine volume in ppt.

d) Density. The density of sea ice is relatively constant, with typical values ranging from 0.88 to 0.92.

e) Impurities. As sea ice melts, impurities, such as brine, are expelled through drainage channels, creating air pockets which may weaken the ice.

A.2.2.2. MECHANICAL PROPERTIES

The mechanical properties of sea ice are dependent on many factors, including its crystallographic structure, temperature, brine volume, and strain rate. Since most of the available strength data were obtained from small sample testing, care should be taken in applying small-scale test data to analyze large-scale field problems.

a) Compressive Strength. The unconfined compressive (crushing) strength of sea ice has been obtained from several laboratory and field tests of small scale samples (Butkovich, 1956; Peyton, 1966; Vaudrey, 1977 and Schwarz 1970.

Sea ice subject to confinement has a much greater apparent strength than the uniaxial strength, but the relationship between unconfined and confined compressive strengths are presently not well established.

The compressive strength varies with strain rate, (Vaudrey, 1977) crystal structure and orientation (Exxon, 1979 and Peyton, 1966) temperature (Tryde, 1979 and Haynes, 1979), salinity (Tryde, 1979 and Peyton, 1966), grain size (Sinha, 1981) and confining stress (Exxon, 1977 and Hausler, 1981). Sample data are given in Figs. 1-6.

b) Tensile Strength. Tensile strength varies with salinity (Dykins, 1971 and Frankenstein, 1967), temperature (Haynes, 1979) and given size and orientation (Peyton, 1966) and is related to fracture toughness (see paragraph h). Sample data are given in Figs. 7-10.

c) Flexural Strength. Flexural strength varies with salinity (Vaudrey, 1977; Schwarz and Weeks, 1977; Dykins, 1971 and Tryde, 1979), stress rate (Sacki, Ozaki and Kubo, 1981) and temperature (Sacki, Ozaki and Kubo, 1981 and Tryde, 1979). Sample data are given in Figs. 10 and 11.

d) Shear Strength. The shear strength of ice as a function of brine volume is presented in Fig. 12. The plot is based on small sample tests by Paige and Lee (1967).

e) Adfreeze Strength. A knowledge of adfreeze strength is important to predict breakout loads when sea ice, frozen to structural surfaces, is subjected to ice movement. Adfreeze strength is a function of salinity (Sackinger and Sackinger, 1977), temperature (Sackinger and Sackinger, 1977), material, stress rate and contact area (Sacki, Ono and Ozaki, 1981). Sample data are given in Figs. 13 and 14.

f) Elastic Modulus. Static measurements of elastic modulus (apparent modulus) are difficult to interpret because of the visco-elastic behaviour of ice when subjected to significant stresses for finite time periods. In dynamic measurements based on rate of wave propagation or resonant vibrations the displacements are small and anelastic effects are removed. The elastic modulus is a function of salinity (Vaudrey, 1977; Langleben and Pounder, 1963; Anderson, 1958 and Dykins, 1971) and temperature (Weeks and Assur, 1967). Sample data are given in Figs. 15-17.

g) Poisson's Ratio. Poisson's ratio is close to 0.3 and is a functicn of temperature and stress rate (Sacki, Ozaki and Kubo, 1981 and Graff, 1977. Figs. 18 and 19).

h) Fracture Toughness. Crack like flaws exist in sea ice and fracture toughness is useful in predicting failure. Fracture toughness is a function of strain rate and grain size (Urake and Yoshitake, 1981, a, b).

A.2.3 RIDGES AND RUBBLE FIELDS

Ridges and rubble fields are random collections of ice pieces. The strength depends on many factors, including: (Prodanovic, 1979 and Weiss, Prodanovic and Wood, 1981)

- mechanical properties of parental ice sheet,
- degree of consolidation (refreezing),
- confining pressure,
- size of ice pieces,
- packing (void ratio),
- temperature, and
- strain rate.

A.2.4 ICEBERGS

Very little information is available on physical and mechanical properties of icebergs. Icebergs calve from glaciers and have properties similar to glacier ice with a large number of cracks. Data on tests on a few bergs are given in Tables 1 and 2.

Table 1: Iceberg Porosity (Ice Engineering, 1981)

Iceberg No.	Percentage Porosity	No. of Measurements
1	0	23
2	1.7	3
3	1.1	4
4	1.2	7
5	2.7	6
6	2.5	5

Table 2: Chemical Analysis (Ice Engineering, 1981)

pH	-5.53		
	0/00		0/00
Flouride	0.044	Sodium	0.19
Chloride	0.4	Sulphate	2.00
Calcium	0.038	Magnesium	1.1

A.2.5 TESTING METHODS

Standardized testing procedures to determine ice parameters so far have not been established. The IAHR Working Group on Ice-Testing Methods could be expected to initiate standardized testing procedures.

A.3. LOADS

A.3.1 GENERAL

All load magnitudes expressed in this section are considered unfactored loads, i.e., depending on the design method that is being used for a particular structure, they may need to be modified to properly represent the design loads (refer A.1.2).

A.3.2 ICE FAILURE MODES

A.3.2.1 FAILURE MODES ON VERTICAL-SIDED STRUCTURES

a) Sheet Ice. Failure modes for sheet ice include the following:

- Buckling;
- crushing;
- ridge building;
- others, depending on the detailed multidimensional strength properties of sea ice, the rate of loading, the total strain, inhomogeneity, flaws and inclusions.

b) First-year Ridges and Rubble Piles. Potential failure modes for first-year ridges and rubble piles include the following:

- Crushing, depending on the relative dimensions of the ridge, and the thickness and strength of the consolidated zone;

- bending in the horizontal plane;
- double-sided shear along vertical planes;
- rubble fields in double-sized shear followed by the formation of a rubble pile in front of the structure with subsequent flow of rubble around the rubble pile and structure.

c) Failure Modes of Multiyear Ridges. Potential failure modes for multiyear ridges may include failure of the ridge beam itself or failure of the associated sheet ice or any combination of these.

- Crushing, depending on the relative dimensions and strength of the ridge;
- Crushing of associated sheet ice;
- Bending failure in the horizontal plane of the ridge and associated sheet ice;
- Buckling of associated sheet ice;
- Double-sided shear of the ridge followed by crushing/buckling of associated sheet ice in between the lines of the shear cracks;
- Vertical bending failure of associated sheet ice in connection with rotation (torsion) of the ridge.

d) Driving Forces. For any failure mode listed above to occur, sufficient force must be transmitted through the general ice cover in the vicinity of the structure (Vivatrat and Kreider, 1981). The available force depends on the continuity of the ice cover and the dimensions of any force collector which might surrounded the ice feature. Thus, the driving force can be expected to vary with location within the general ice cover.

A.3.2.2 FAILURE MODES ON SLOPING-SIDED STRUCTURES

In general, sloping-sided structures induce vertical deflection of the ice feature, thus producing bending failures. Other failure modes are possible, however, they depend on the relative strength and the degree of continuity between the ice and the structure.

a) Sheet Ice. Potential failure modes for sheet ice include the following:

- Adhesion bond failure by breaking the bond formed when ice grows adjacent to a structure;
- crushing failure if the adhesion bond strength is large relative to the crushing strength of the ice, or a previous pile-up

against the sloping structure presents an essentially vertical face to the approaching sheet ice;
- bending failure characterized by ice riding up the structure and failing in flexure at some distance away from the structure;
- buckling;
- ridge building.

b) First-year Ridges and Rubble Fields. Potential failure modes for first-year ridges and rubble fields include the following:

- Bending failure in the vertical plane;
- double-sided shear failure along vertical planes;
- rubble field failures initially in a form of double-sided shear followed by the formation of a rubble pile in front of the structure and subsequent flow of ice around the structure.

c) Multiyear Ridges. A potential failure mode for multiyear ridges is bending in the vertical plane.

A.3.2.3 ICE CRUSHING LOADS

The horizontal force exerted by ice crushing against a structure may be calculated as:

$$F = I\, f_c\, C_x\, Dt \qquad (2)$$

where F = horizontal ice force;

I = indentation factor;

f_c = contact factor;

C_x = unconfined compressive strength of the ice;

D = diameter or width of the structure at the region of ice contact; and

t = ice thickness.

a) Indentation Factor. The indentation factor, I, depends on:

- crystallographic structure of the ice;

- multiaxial strength of the ice;
- strain rate of the ice; and
- geometry of the interaction between the ice and the structure.

The strain rate $\dot{\varepsilon}$ for the ice is a function of the ice approach velocity, v and the structure dimension, D. Exxon (1979) used the following relationship for the case of a gravel island in shallow water in the Beaufort Sea:

$$\dot{\varepsilon} = \frac{v}{2D} \qquad (3)$$

Alternative strain rate expressions have been suggested (Michel and Toussaint, 1977 and Ponter, Palmer, Goodman, Ashby, Evans and Hutchinson, 1982).

Indentation factors have been developed from plasticity analysis by assuming ductile behaviour of the ice (Ralston, 1978). Laboratory data analyzed in these studies have verified factors derived from plasticity analyses in which measured identation pressures divided by calculated identation factors agree with ice strength data over a range of strain rates. Figure 20 gives examples of indentation factors for laboratory-grown columnar fresh water ice. These curves have also been used for columnar saline ice. For granular sea ice, Exxon (1979) used an indentation factor of 1.2 for a large structure/sheet ice thickness ratio.

b) Contact Factor. The contact factor, f_c, depends on:

- ice movement rate;
- local geometric effects; and
- active defensive mechanisms.

Peyton's (1968) analysis of ice forces on Cook Inlet structures suggests a maximum value of 0.55 for the product of I x f_c. Assuming an indentation factor of 1.2, the calculated contact factor of 0.45 appears to be consistent with the relatively continuous ice movements and nonfrozen-in conditions of Cook Inlet and falls within the range of contact factors presented by Michel and Toussaint (1977).

Ice strength increases with decreasing temperature. The crushing load is also dependent on the type of failure whether it is dectile or brittle with higher loads for ice failing in a ductile mode (Taylor, 1981) and higher pressures could occur in warm ice. Failure could be limited to the interface, across the plane of the ice sheet by flow around the structure (Michel, 1981), shear-type double wedge failure (Taylor, 1981 and Nakajima, Koma and Inoue, 1981), by plastic flow or flaking (Kry, 1981).

A.3.2.4 SHEET ICE BENDING LOADS

For sloping-sided structures, sheet ice moving against a structure may fail in bending and repeated fracture.

A limit analysis solution for sheet ice bending failure against a conical structure has been developed by Ralston (1979). The solution includes force components for ice breaking and ice ride-up on the structure.

These forces are based on plastic limit analysis. Other mechanisms of failure should be considered (creep, fracture) depending on the nature of the ice environment.

Under certain circumstances, such as very wide structures or steep angles, alternate force prediction methods may be necessary. For example, APOA Project No. 87 (1975) presents an elastic solution for sheet ice bending. Model studies may be used to evaluate ice loading. When using elastic theory, the use of corrective factors, which may be developed from model studies, should be considered to account for interactive forces or plastic action.

A.3.2.5 SHEET ICE BUCKLING LOADS

When the ice thickness is small, the ice sheet may buckle rather than crush or bend. Elastic solutions for buckling loads may be obtained from Sodhi and Hamza (1977), Sodhi (1979), and Wang (1978). The buckling load, P, is given as (Sodhi, 1977):

$$P = KL^3 [b/L + 3.32/(1 + b/4L)] \qquad (4)$$

where b = Width of structure

$L = (D/K)^{1/4}$ = characteristic length

$b = Et^3/(12\,(1-\nu^2))$

E = Elastic modulus of ice sheet

ν = Poisson's ratio.

K = Modulus of water support (specific weight of water).

A.3.2.6 FIRST-YEAR RIDGE LOADS

Loads from first-year ridges may develop either from crushing or shearing (Croasdale, 1978). The crushing strength of ice

rubble is assumed to be considerably lower than the crushing strength of individual ice pieces making up the rubble.

If partial consolidation of the ice rubble is anticipated, i.e., refreezing of the ice rubble pieces near the zone of contact with the seawater, the refrozen zone may be treated as a solid ice sheet with a thickness that is equivalent to that of the frozen zone. Loads from partially consolidated ridges may then be calculated as the combination of a sheet ice force, from failure of the sheet ice either in crushing or bending, and the force anticipated to shear or crush the ice rubble taking into account the respective stiffnesses of the ice sheet and the rubble. In combining the force components, it should be kept in mind that the peak values for the individual force components may not occur simultaneously. Analytical solutions for consolidated and unconsolidated ridges based on plasticity theory were developed by Prodanovic (1981).

A.3.2.7 FIRST-YEAR RUBBLE FIELD LOADS

Rubble fields have mechanical properties similar to first-year ridges, except that their lateral extent exceeds the finite width of the ridge. It is anticipated that, due to the size of a rubble field, loads will develop from ice rubble crushing when making contact with the structure.

A.3.2.8 MULTIYEAR RIDGE LOADS

Multiyear ridges may either fail by crushing, bending, or shearing against the structure or may come to rest at the structure without failing due to an insufficient driving force. The preferred mode of long multiyear ridge failure is bending, since this may result in the lowest load from such a ridge. For short ridges the force required to crush the sheet behind the ridge may be smaller (Rajansky and Gerwick, 1981).

In determining ridge forces, the geometry, ice properties, velocity and orientation of the ridge should be considered, along with the environmental force available to drive the ridge against the structure.

A.3.2.9 ADFREEZE FAILURE LOADS

In the absence of significant or continuous ice movement, a bond may be established between the ice and the structure. Also, broken ice pieces may adhere to the surface of smooth-sided structures. The force necessary to break the adhesion bond should be evaluated. Depending on the relative

value of the bond strength to other ice strengths, the bond may fail first, thus allowing other failure modes such as bending to occur. APOA Project No. 57 (1973) contains a force computation procedure for adhesion failure forces.

Heat flow through a structure and its effect upon reconsolidation and adfreeze of ice fragments adjacent to the structure should be taken into account in estimating the strength and thickness of ice adhesion to the structure.

Floating ice covers frozen to the structure induce vertical loads due to change in the water level. The load is a function of ice thickness, and properties, structure dimention and rate of change of water level (Frederking, 1979).

A.3.2.10 LOCAL ICE CONTACT PRESSURES

The pressure distribution of ice features in contact with a structure should be determined for local structural design considerations. Ice contact failure pressures are anticipated to be considerably larger than the unaxial ice strength because of the confinement of the ice in local contact with the structure.

Ship classification society rules for ice strengthening, such as contained in the Canadian Arctic Shipping Pollution Prevention Regulations, and experiences with ice strengthened ships and ice-breakers provide some guidance for contact pressure distribution. Because these regulations are intended for ships, not all provisions will apply for fixed structures.

A.3.2.11 DYNAMIC EFFECTS

Inertial ice forces can occur when ice features of large mass at low velocities, e.g., ice islands, or smaller features at high velocities, e.g., summer storm driven floes, encounter a structure. The forces necessary to arrest or deflect the ice features should be evaluated and compared to the forces necessary to fail the ice features considering ice strength parameters corresponding to appropriate loading rates. Added mass effects from the water should be allowed for (Sorensen, 1978).

In conditions associated with continuous ice movements at relatively high velocity, ice fracture may occur at uniform time intervals. The fracture frequency or periodicity may be at or close to a fundamental frequency of the structure, resulting in the phenomenon of "ratcheting" reported by Matlock, Dawkins and Panak (1969) for vertical-sided

indentors. A similar phenomenon is theoretically possible in the case of cone-shaped structures also (Sorensen, 1978). Some of the theoretical considerations associated with these pehnomena are discussed by Maattanen (1980, 1981) and Eranti, Hayes, Maattanen and Soong (1981).

A.3.3 ARTIFICIAL ISLANDS AND WIDE STRUCTURES

Islands (includes wide structures) may be made from gravel, ice, or other suitable construction materials. Their main geometric characteristics are their large size (diameter) and their reliance on gravitational forces to develop foundation resistance against ice loads.

Sheet ice crushing loads may be calculated from Eq. 2. In selecting unconfined ice crushing strengths, values should be selected that correspond to the anticipated ice movement rates, brine volume and temperature range. Strain rates may be determined from predicted or measured ice movement rates for a given site. Data on indentation factors and ice-structure contact factors required to evaluate Eq. 2 are given in Exxon (1979) for low ice movement rates. Increased structure width generates the possibility of nonsimultaneous failure. The resulting averaging of variations across the width leads to reduced stresses (Kry, 1978, 1980).

The selection of D, the island diameter, depends on the prevailing ice conditions in contact with the island. If an island is surrounded by ice rubble, the load required to fail the rubble should be compared against the required load to cause sheet ice crushing against the rubble field. In the latter case, Eq. 2 should be evaluated with D representing the total width as measured across the island and rubble field. The load required to fail the rubble field may also be assessed by Eq. 2 except that t, the thickness of the ice, should represent the thickness of the consolidated portion of the rubble field. Since there are difficulties in determining the thickness or extent of rubble fields, care should be exercised in interpreting the forces due to rubble fields. The smallest of the calculated loads should be selected as the load from which to determine the design load.

If the ice rubble is grounded around the island, not all of the load exerted against the rubble field will be transmitted to the island, some if it will be dissipated by the foundation in contact with the rubble. Kry (1977) calculated the net island load in proportion to the net weight of the rubble acting on the sea floor to the weight of the island. However, other methods may be used to assess the portion of the total load that is removed by the rubble pile.

A.3.4 IMPACT OF ICE FLOES AND ICEBERGS

When a large floe or iceberg collides with a massive structure, the failure is at the contact zone which will increase in size and result in increasing resisting force. The kinetic energy of the ice feature reduces as it decelerates till an equilibrium is reached with the resisting force (Cammaert and Tsinker, 1981; Neill, 1970). If the floe size to thickness is large, bending or buckling could occur. Small ice features could be carried by the waves and impact on the structure.

A.3.5 FLOATING STRUCTURES

Information on ice forces on ships could be adapted to predict the forces on floating structures (Major, Borenger, and Laurie, 1975; Noble, Tam, Menon and Bayly, 1979; Popov, Faddeev, Kheisin and Yakovlev, 1968; and Enkvist, Varsta and Riska, 1979). The effect of mooring and other station keeping systems should be considered.

A.3.6 DRIVING FORCES - WIND AND CURRENT

The force that the ice feature exerts on the structure is limited to kinetic energy of the feature, the force required to form ridges (Parmerter and Coon, 1973) and the wind and current forces on the feature (Croasdale and Marcellus, 1981; J.V. Danys, 1977; Bruun, 1981 and Vivatrat and Kreider, 1981). The wind and current drag forces, F, per unit width, can be expressed as:

$$F = PCFV/V/$$

where P = density of air/water

C = drag coefficient for wind/current,

F = Fetch length, and

V = Relative velocity of ice and wind/current.

A.3.7 RELATED PUBLICATION

API Bulletin for Planning, Designing and Constructing Fixed Offshore Structures in Ice Environments.

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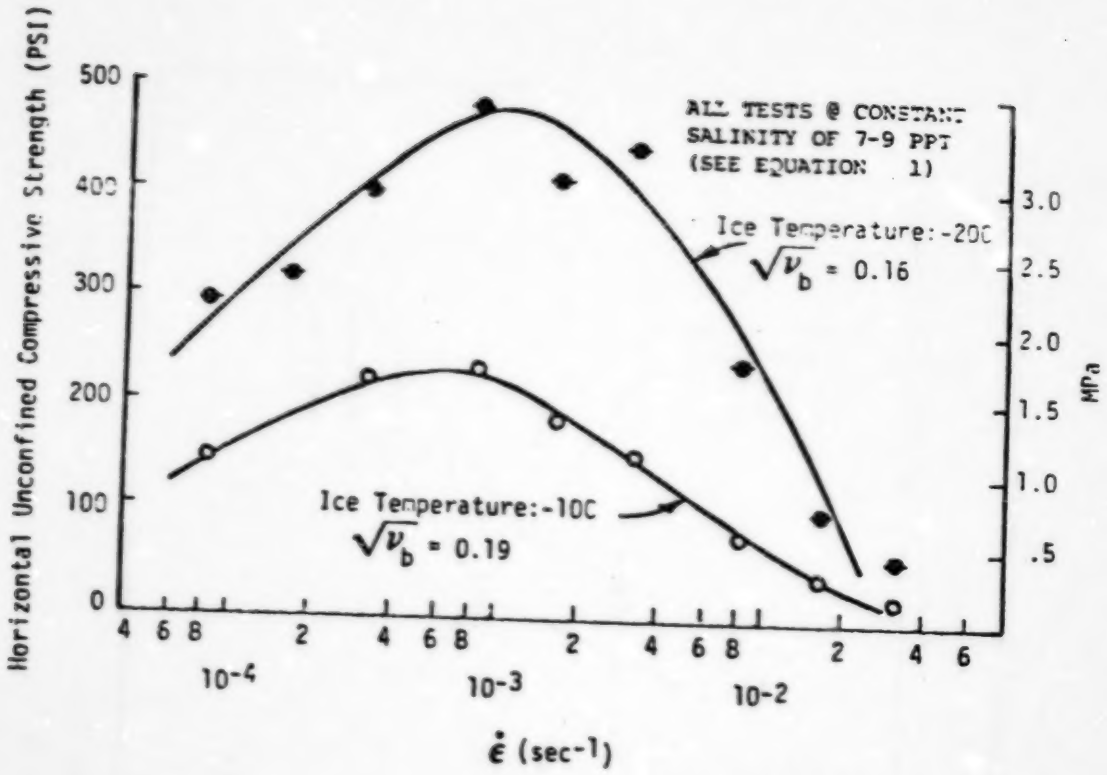


FIG. 1 Horizontal Compressive Strength Versus Strain Rate for Random C-Axis Oriented Sea Ice (Vaudrey, 1977)

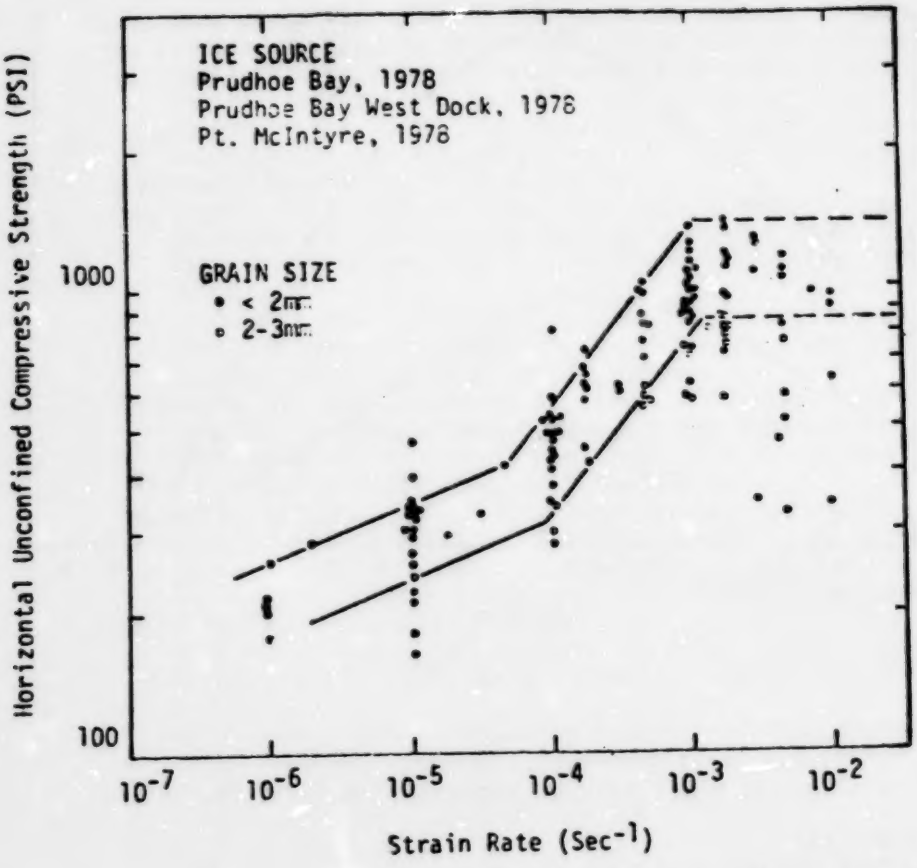


FIG. 2 Compressive Strength of Granular Sea Ice as a Function of Strain Rate (Exxon, 1979)

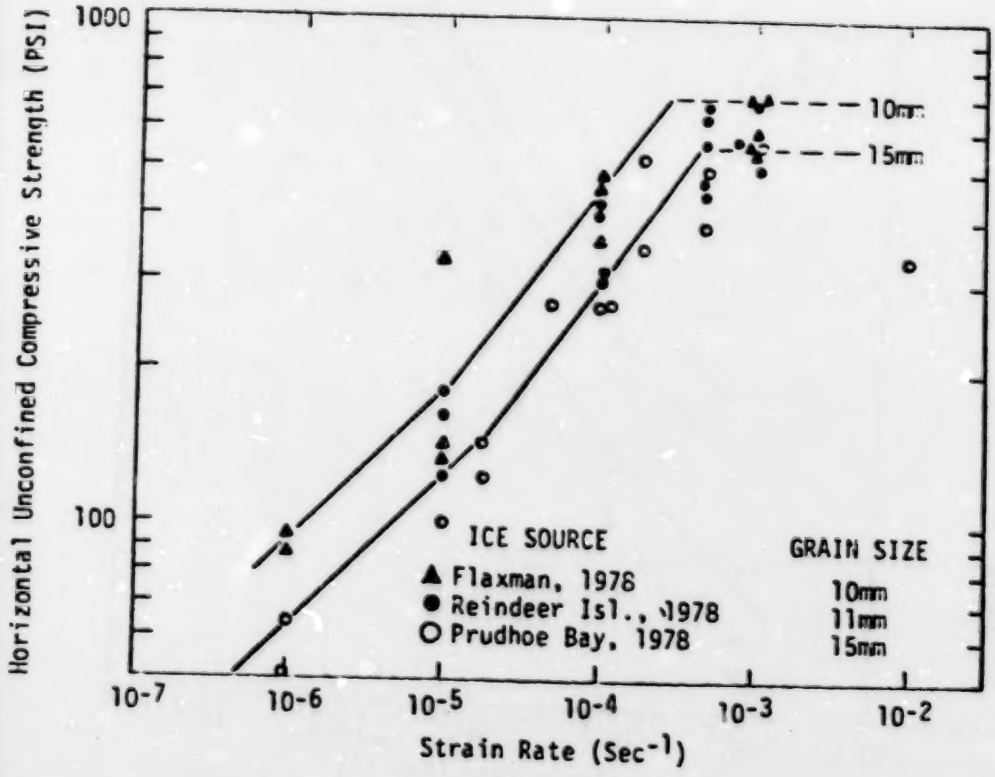


FIG. 3 Compressive Strength of Unoriented-Columnar Sea Ice as a Function of Strain Rate (Exxon, 1979)

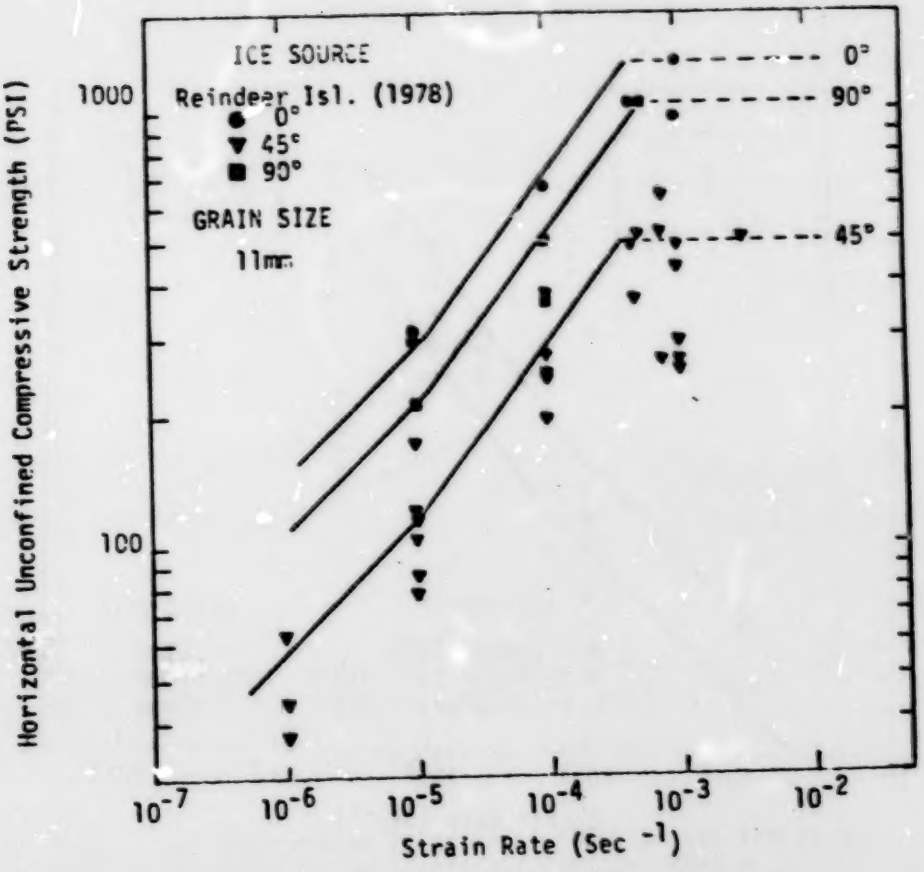


FIG. 4 Compressive Strength of Oriented-Columnar Sea Ice as a Function of Strain Rate (Exxon, 1979)

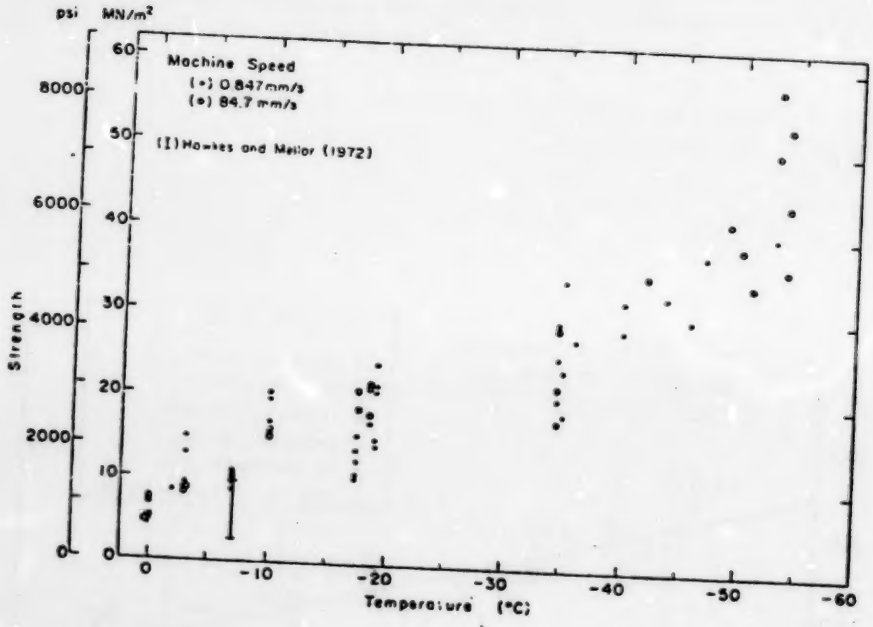


Figure.5 Compressive strength as function of temperature (Haynes, 1979)

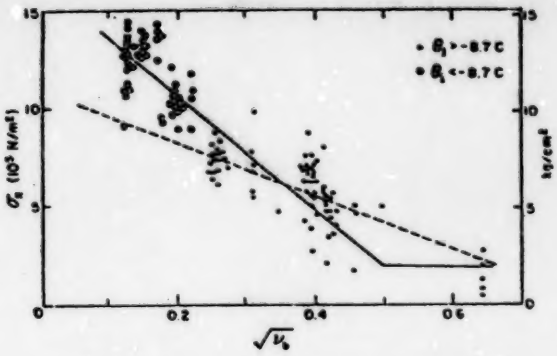


Figure 6 Maximum strength vs. square root of brine volume. The dashed line represents on interpretation of the results which speculates reinforcement by solid at low temperatures (Peyton, 1968)

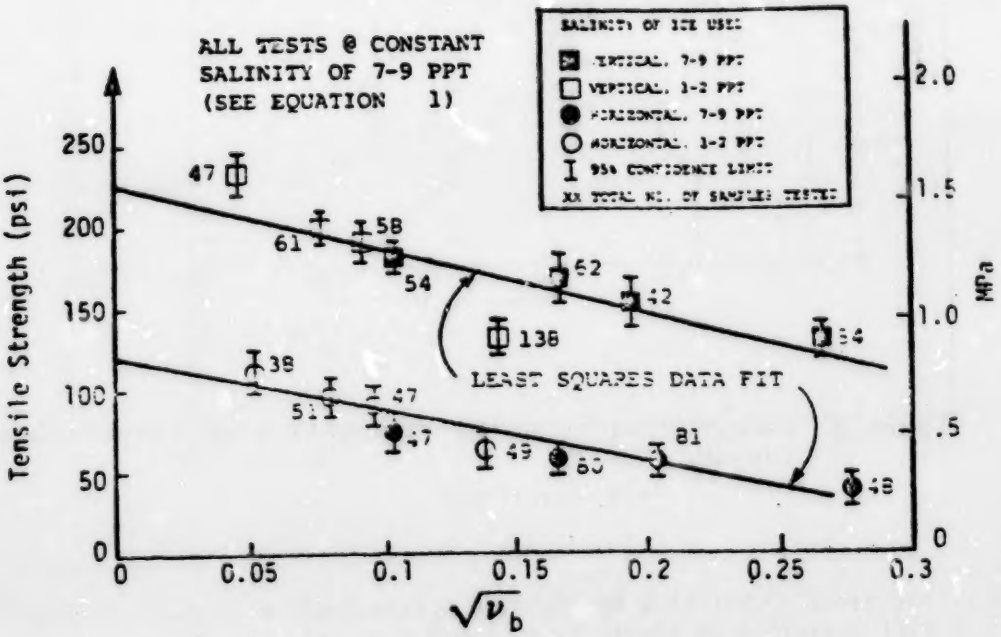


FIG. 7 Sea Ice Tensile Strength as a Function of the Square Root of Brine Volume (Dykins, 1971)

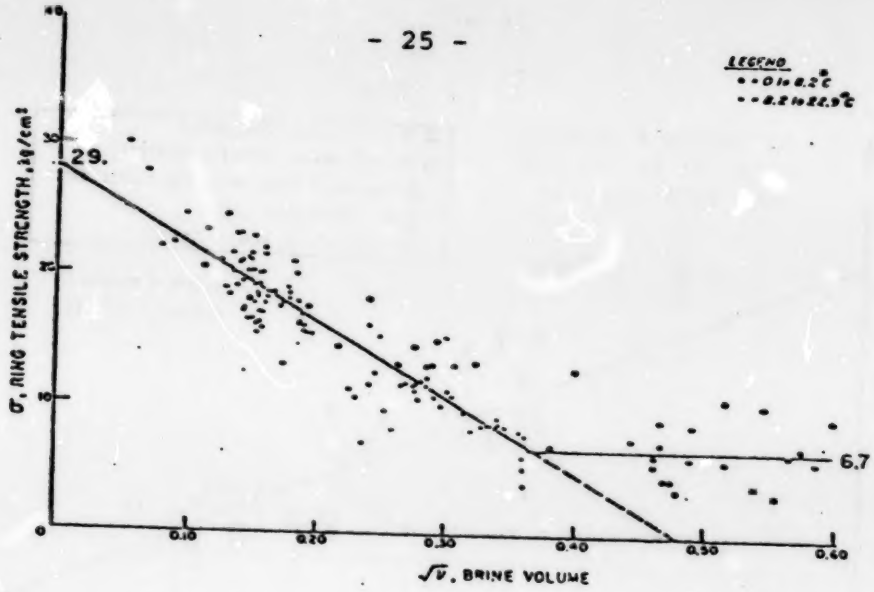


Figure 8 Ring tensile strength vs. square root of the brine volume, sea ice. (Frankenstine, 1967)

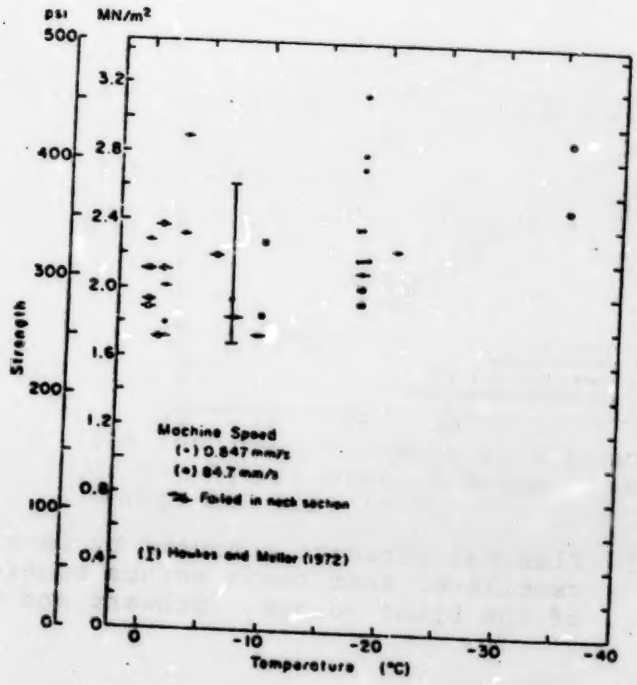


Figure 9 Tensile strength as a function of temperature (Haynes, 1979)

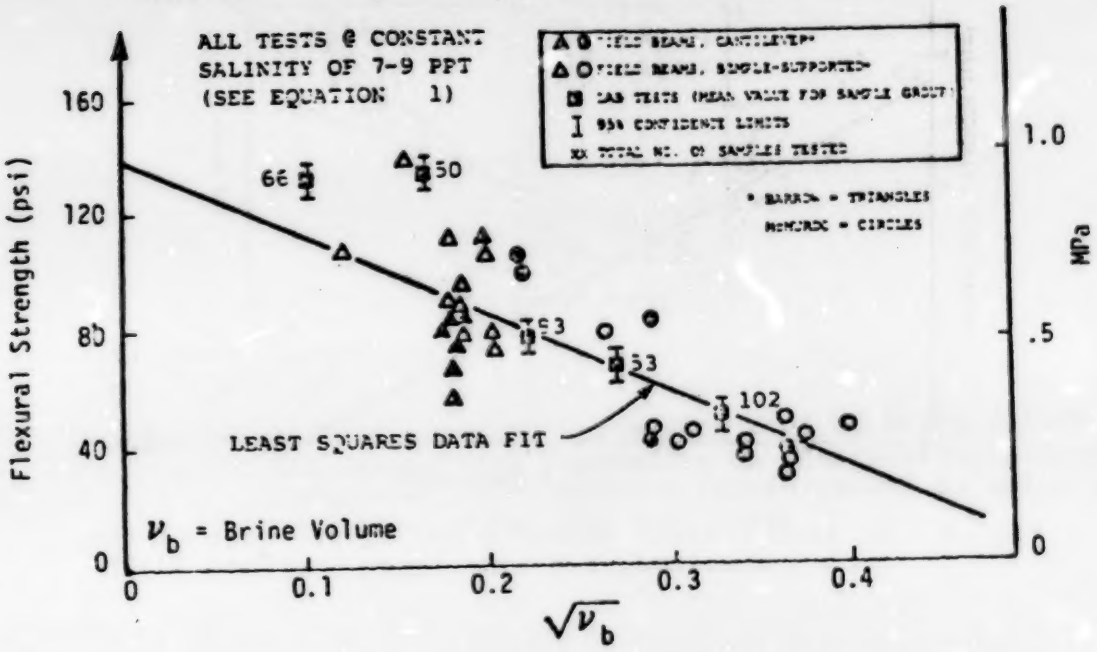


FIG. 10 Sea Ice Flexural Strength as a Function of the Square Root of Brine Volume (Vaudrey, 1977)

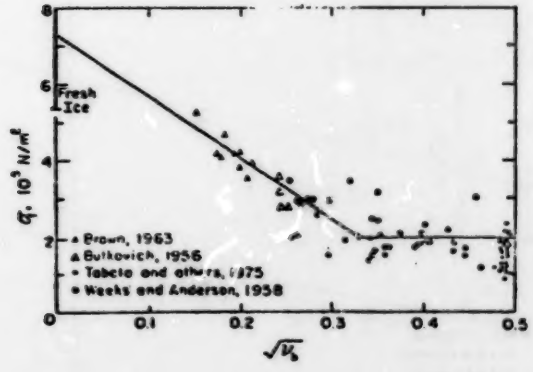


Figure 11 Flexural strength measured by in-situ cantilever beam tests versus square root of the brine volume. (Schwarz and Weeks, 1977)

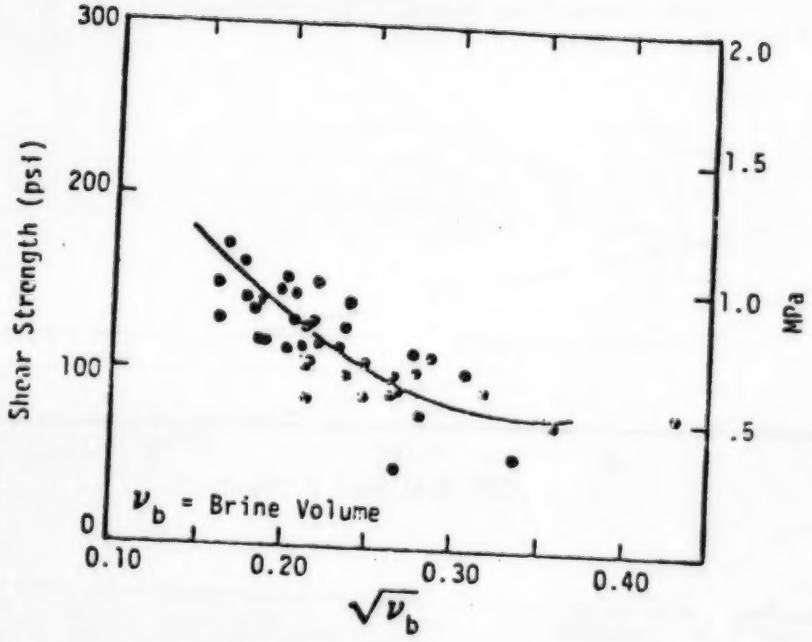


Figure 12 Sea Ice Shear Strength as a Function of the Square Root of Brine Volume (Paige and Lee, 1967)

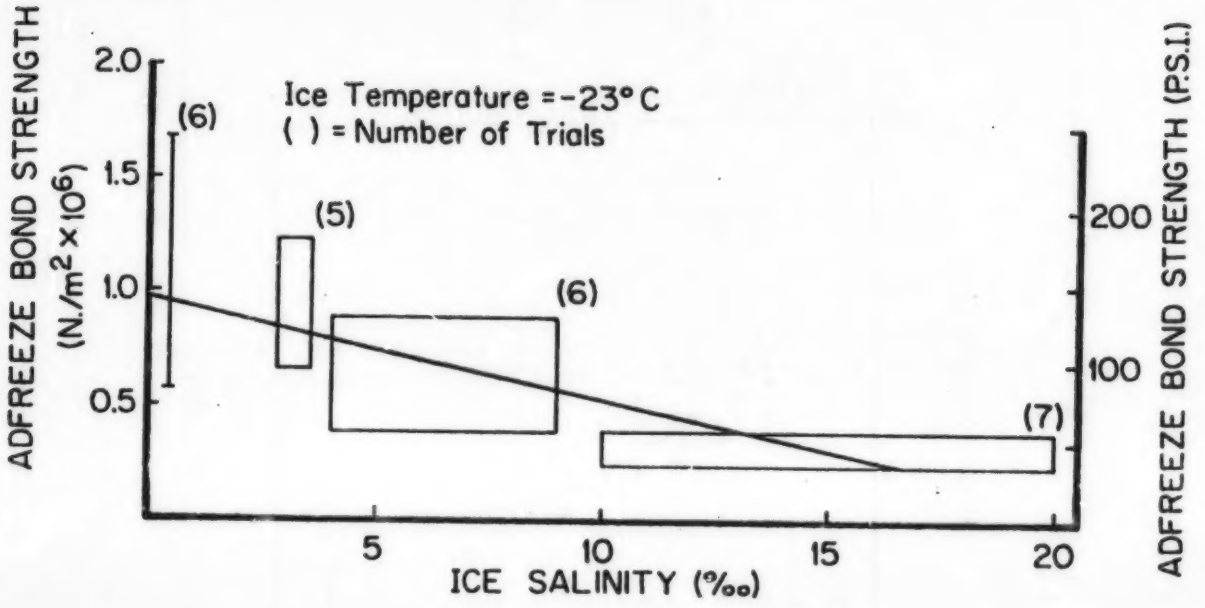


Figure 13 Adfreeze bond of saline ice to steel as a function of salinity. (Sackinger and Sackinger, 1977)

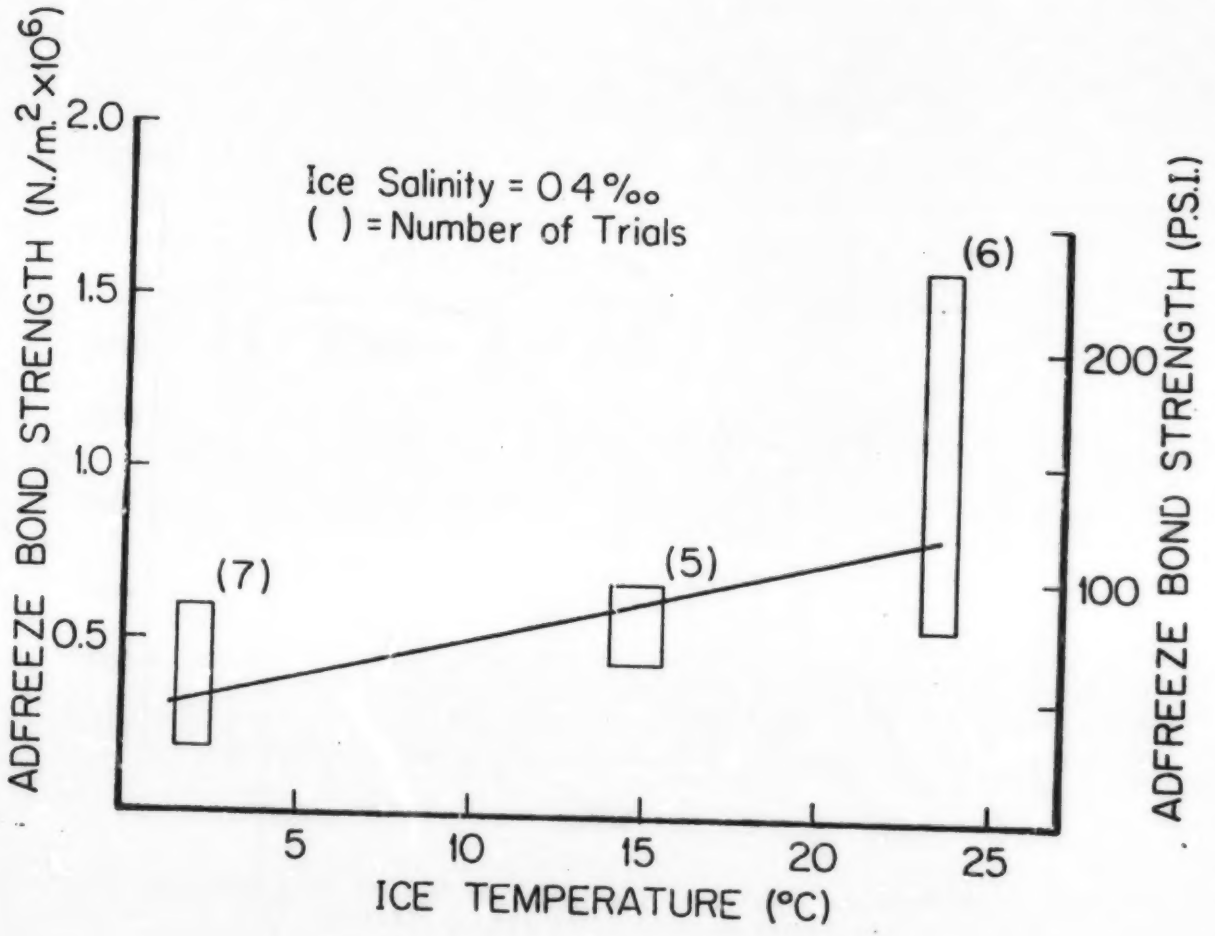


Figure 14 Adfreeze bond of ice to steel as a function of temperature. (Sackinger and Sackinger, 1977)

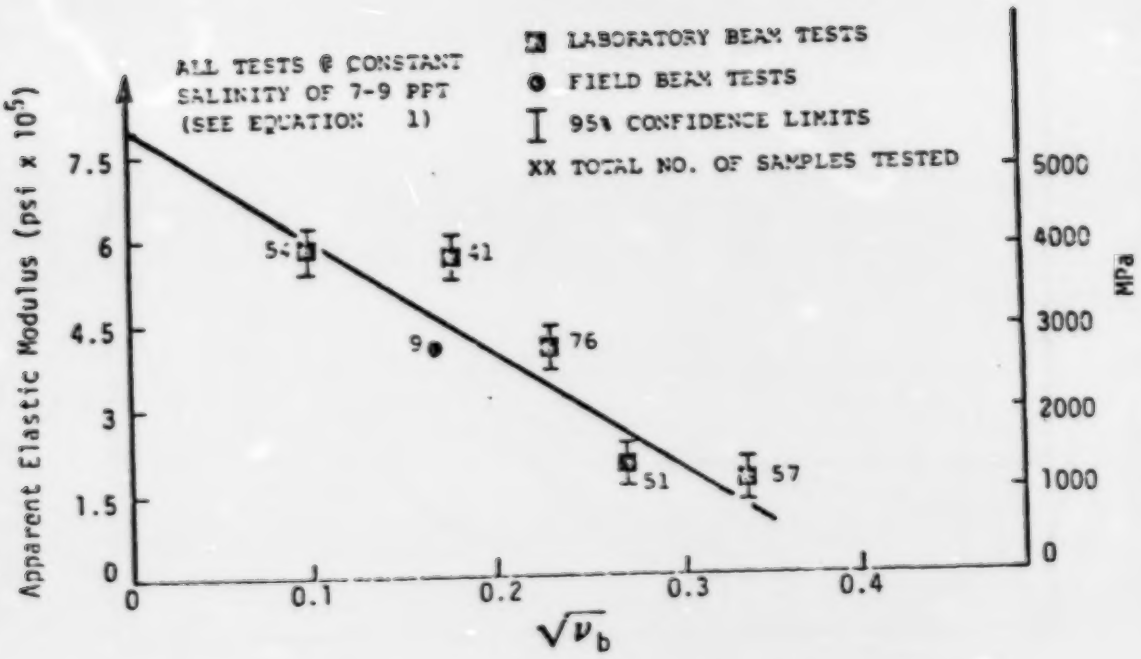


FIG. 15 Apparent Modulus of Elasticity of Sea Ice as a Function of the Square Root of Brine Volume (Vaudrey, 1977)

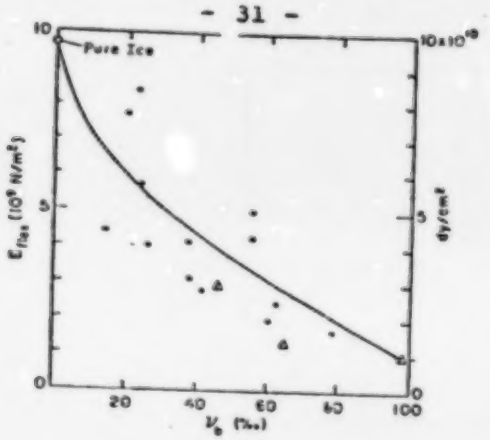


Figure 16 Elastic modulus of sea ice as determined by seismic techniques versus brine volume (Schwarz and Weeks, 1977)

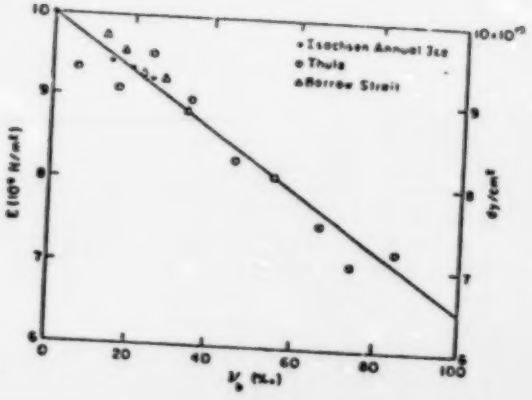


Figure 17 Elastic modulus of cold arctic sea ice as determined by small specimen tests versus brine volume (Langleben and Ponder)

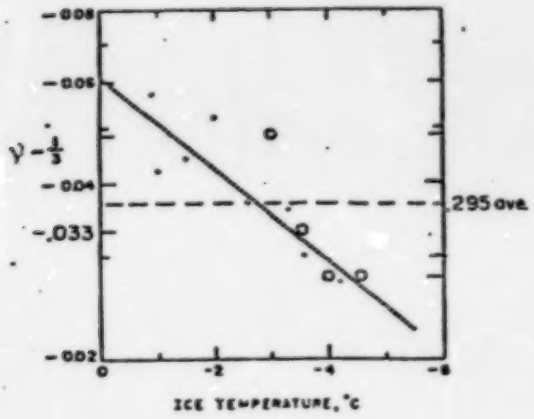


Figure 18 Poisson's ratio γ plotted as γ minus one-third versus mean ice temperature. Position of average 0.295 for Poisson's Ratio is shown on right ordinate (Graft, 1977)

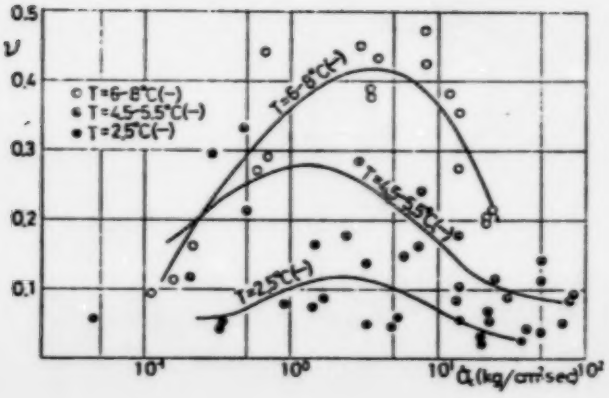


Figure 19 Experimental results of Poisson's ratio for porous sea ice (Saeki, Ozaki and Kubo, 1981)

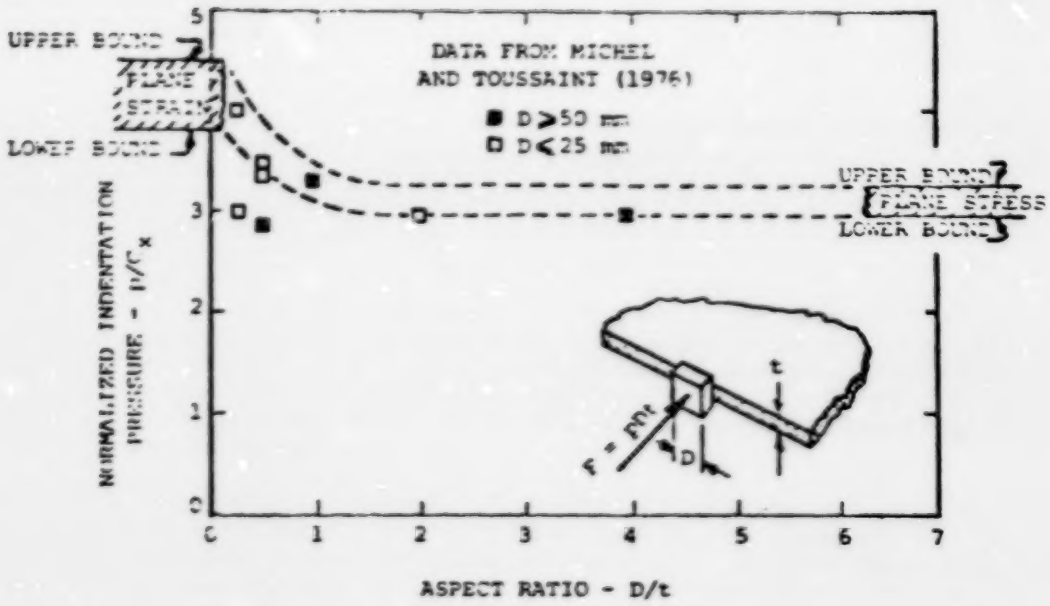


FIG. 20 Comparison of Computed Bounds for Indentation Pressure with Test Data for Columnar Ice (Ralston, 1978)

APPENDIX B: FATIGUE

APPENDIX B: FATIGUE

B.1 STRESSES TO BE CONSIDERED

(a) For nodal joints, i.e. chord to brace connections, the stress to be used for design purposes is the range of idealized hot spot stress adjacent to the weld toe. The idealized hot spot stress is defined by linear extrapolation to the weld toe of the principal stress at A and B in Figure B.1. (In the region A-B it has been found that the stress distribution is approximately linear).

It should be noted that this idealized hot spot stress may be considerably lower than the maximum stress since it does not include the effect of the rapid increase in stress adjacent to the weld toe. However, the relevant design S-N curve (Curve T, Figure B.4) is consistent with the definition of idealized hot spot stress given above.

The calculation of hot spot stress may be undertaken in a variety of ways, e.g. by finite-element analysis, physical model studies or by use of semi-empirical parametric formulae. The position of the hot spot in relation to the crown and saddle will be determined by the first two methods but not necessarily in all cases by parametric equations. If the position of the hot spot stress cannot be derived by the latter method then the value generated must be applied to all points around the periphery; at least 8 points should be considered in the analysis.

Parametric formulae for the calculation of hot spot stresses should be used with caution and subject to the agreement of the certifying authority. It has been shown that, in some circumstances, such formulae can predict stresses up to at least 25 percent lower than the idealized hot spot stresses which exist. The situation may be even worse if the formulae are used outside their strict ranges of validity. The certifying authority may require hot spot stresses calculated by means of parametric formulae to be modified by a safety factor in order to allow for these errors.

(b) Non-nodal Joints

For all other types of joints (e.g. welded stiffeners or attachments) the joint classifications and corresponding S-N curves take into account the local stress concentrations created by the joints themselves and by the weld profile, and the design stresses can therefore be regarded as the nominal stresses adjacent to the weld under consideration. However, if the joint is situated in a region of stress concentration resulting from the gross shape of the structure; this must be

taken into account in calculating the nominal stress. For example, for the weld shown in Figure B.2a the nominal stress would be the tensile stress, S, on the cross-section, but for the weld shown in Figure B.2b it would be $K_t S$, where K_t is the stress concentration factor caused by the hole. (This results from the fact that at point X the stress acting on the weld is $K_t S$).

B.2 JOINT CLASSIFICATION

For the purposes of fatigue design, welded joints are divided into several classes, each with a corresponding design S-N curve (see paragraph B.3a). All nodal joints are assumed to be in Class T, but other types of joints may fall in one of the eight classes specified in Figure B.3, i.e. B, C, D, E, F, F2, G or W depending upon:

i) the geometrical arrangement of the detail;

ii) the direction of the fluctuating stress relative to it; and

iii) the method of manufacture and inspection.

Each constructional detail in a member subject to fluctuating stresses must, where possible, be placed in its relevant joint class in accordance with the criteria given in Table B.1. Otherwise the detail may be dealt with in accordance with paragraph B.2a.

It should be noted that, in any welded joint, there are several locations at which fatigue cracks may potentially develop, e.g. at the weld toe in each of the two parts joined, at the weld ends, and in the weld itself. Each must be classified separately. For example, in the case of members or elements connected by load-carrying fillet welds or partial penetration butt welds cracks may initiate either in the parent metal (at one of the weld toes) or in the weld throat; both possibilities must be checked by defining the appropriate classification and the corresponding stress range.

(a) Unclassified Details

Details which are not expressly classified in Table B.1 should be treated as Class G, or Class W for load-carrying weld metal, unless a higher classification can be justified either by reference to published experimental work or by carrying out special tests. Such tests should be sufficiently extensive to allow the design S-N curve to be determined in the manner used for the standard classes.

B.3 DESIGN STRESSES

(a) Basic Design S-N Curves

The basic design S-N curves for each of the joint classes are shown in Figures B.4 and B.5. For joints which are either:

(i) in air, or
(ii) exposed to sea water, but adequately protected from corrosion

the design curves consist of linear relationships between Log S and Log N. They are based upon a statistical analysis of appropriate experimental data and may be taken to represent two standard deviations below the mean line. In practice this approaches a 97.7 percent probability of survival.

Thus the S-N curves are of the form:

$$\text{Log } N = \text{Log}(K_1) - d.\text{Log}(\sigma) + m.\text{Log}(S)$$

where

N is the predicted number of cycles to failure under stress range S;

K_1 is a constant relating to the mean S-N curve;

d is the number of standard deviations below the mean;

Log σ is the standard deviation of Log N;

m is the inverse slope of the S-N curve.

The relevant values of these terms are shown in Table B.1. This also shows the value of K_2, where

$$\text{Log } K_2 = \text{Log } K_1 - 2.\text{Log}(\sigma)$$

which is therefore relevant to the basic design curves (i.e. for d = 2).

(b) Modifications to Basic S-N Curves

(i) Unprotected joints in sea water
For unprotected joints exposed to sea water the basic S-N curve is reduced by a factor of 2 on life for all joint classes. In addition the correction relating to the numbers of small stress cycles (see paragraph B.3c) is not applicable.

(ii) Effect of plate thickness

The fatigue strength of welded joints is to some extent dependent on plate thickness, strength decreasing with increasing thickness. The basic S-N curves relate to thicknesses as follows:

Tubular joints (Class T)	32mm
Non-tubular joints	up to 22mm

For joints of other thicknesses correction factors on life or stress have to be applied. The correction on stress range is of the form:

$$S = S_B . (t_o/t)^{1/4}$$

where

S is the fatigue strength of the detail under consideration;

S_B is the basic fatigue strength of that detail;

t is the thickness;

t_o is the thickness relevant to the basic S-N curve.

Values of the correction factors for particular thickness ranges are shown in the table below:

TABLE B.2

Thickness range (mm)	Tubular joints (Class T)		Non-tubular joints	
	Factor on stress	Factor on life	Factor on stress	Factor on life
Up to 22	1.10	1.32	1.0	1.0
22 - 32	1.0	1.0	0.91	0.75
32 - 45	0.92	0.77	0.83	0.58
45 - 60	0.85	0.62	0.78	0.47
60 - 80	0.79	0.50	0.72	0.38
80 - 100	0.75	0.42	0.68	0.32

(iii) Weld improvement

For welded joints involving potential fatigue cracking from the weld toe an improvement in strength by a factor of 2.2 on life (equivalent to

30 percent on stress) can be obtained by controlled local machining or grinding of the weld toe. This is normally carried out either with a rotary burr or by disc grinding. The treatment should produce a smooth concave profile at the weld toe with the depth of the depression penetrating into the plate surface to at least 0.5mm below the bottom of any visible undercut (see Figure B.6). The undercut should be such that the maximum depth required should not exceed 2mm or 5 percent of the plate thickness. (Note: the benefit of grinding may be claimed only for welded joints which are adequately protected from sea water corrosion).

(c) Treatment of Low Stress Cycles

Under constant amplitude stresses there is a certain stress range, which varies both with the environment and with the size of any initial defects, below which an indefinitely large number of cycles can be sustained. In air and sea water with adequate protection against corrosion, and with details fabricated in accordance with these procedures, it is assumed that this 'non-propagating stress range' (S_0) is the stress corresponding to $N = 10^7$ cycles; relevant values of S_0 are shown in Table B.1.

When the applied fluctuating stress has varying amplitude, so that some of the stress ranges are greater and some less than S_0, the larger stress ranges will cause enlargement of the initial defect. This gradual enlargement reduces the value of non-propagating stress range below S_0. Thus, as time goes on, an increasing number of stress ranges below S_0 can themselves contribute to the further enlargement of the defect. The final result is an earlier fatigue failure than could be predicted by assuming that all stress ranges below S_0 are ineffective.

An adequate estimate of this behaviour can be made by assuming that all levels of stress are damaging but the number of the lower stresses (i.e. with $S < S_0$) actually applied are reduced by a factor $(S/S_0)^2$. This is equivalent to assuming that the S/N curve has a change of slope from m to (m + 2) at $N = 10^7$ cycles, but with no change in the number of cycles applied. (Note: this correction does not apply in the case of unprotected joints in sea water).

(d) Design Procedure

For each potential crack location the long term distribution of relevant stress ranges (as defined above) is established and the probable fatigue life is estimated by consideration of cumulative damage. It is required that:

$$n_1/N_1 + n_2/N_2 + \ldots = \sum n_i/N_i < 1.0$$

where

n_i are the expected numbers of cycles of the various stress ranges, S_i, in the design spectrum which are assumed to occur in the design life of the structure,

and

N_i are the corresponding numbers of cycles to failure under constant amplitude loading obtained from the relevant S-N curve.

TABLE 8.1: Details of S-N Curves

Class	K_1	$\text{Log}_{10}\ k_1$	$\text{Log}_e\ K_1$	m	Standard deviation, σ Log_{10}	Standard deviation, σ Log_e	K_2	S_o N/mm^2
B	2.343×10^{15}	15.3697	35.3900	-4.0	0.1822	0.4194	1.01×10^{15}	100
C	1.082×10^{14}	14.0342	32.3153	-3.5	0.2041	0.4700	4.23×10^{13}	78
D	3.988×10^{12}	12.6007	29.0144	-3.0	0.2095	0.4824	1.52×10^{12}	53
E	3.289×10^{12}	12.5169	28.8216	-3.0	0.2509	0.5777	1.04×10^{12}	47
F	1.726×10^{12}	12.2370	28.1770	-3.0	0.2183	0.5027	0.63×10^{12}	40
F2	1.231×10^{12}	12.0900	27.8387	-3.0	0.2279	0.5248	0.43×10^{12}	35
G	0.566×10^{12}	11.7525	27.0614	-3.0	0.1793	0.4129	0.25×10^{12}	29
W	0.368×10^{12}	11.5662	26.6324	-3.0	0.1846	0.4251	0.16×10^{12}	25
T	4.787×10^{12}	12.680	29.197	-3.0	0.265	0.6112	1.41×10^{12}	52*

* Idealized hot spot stress

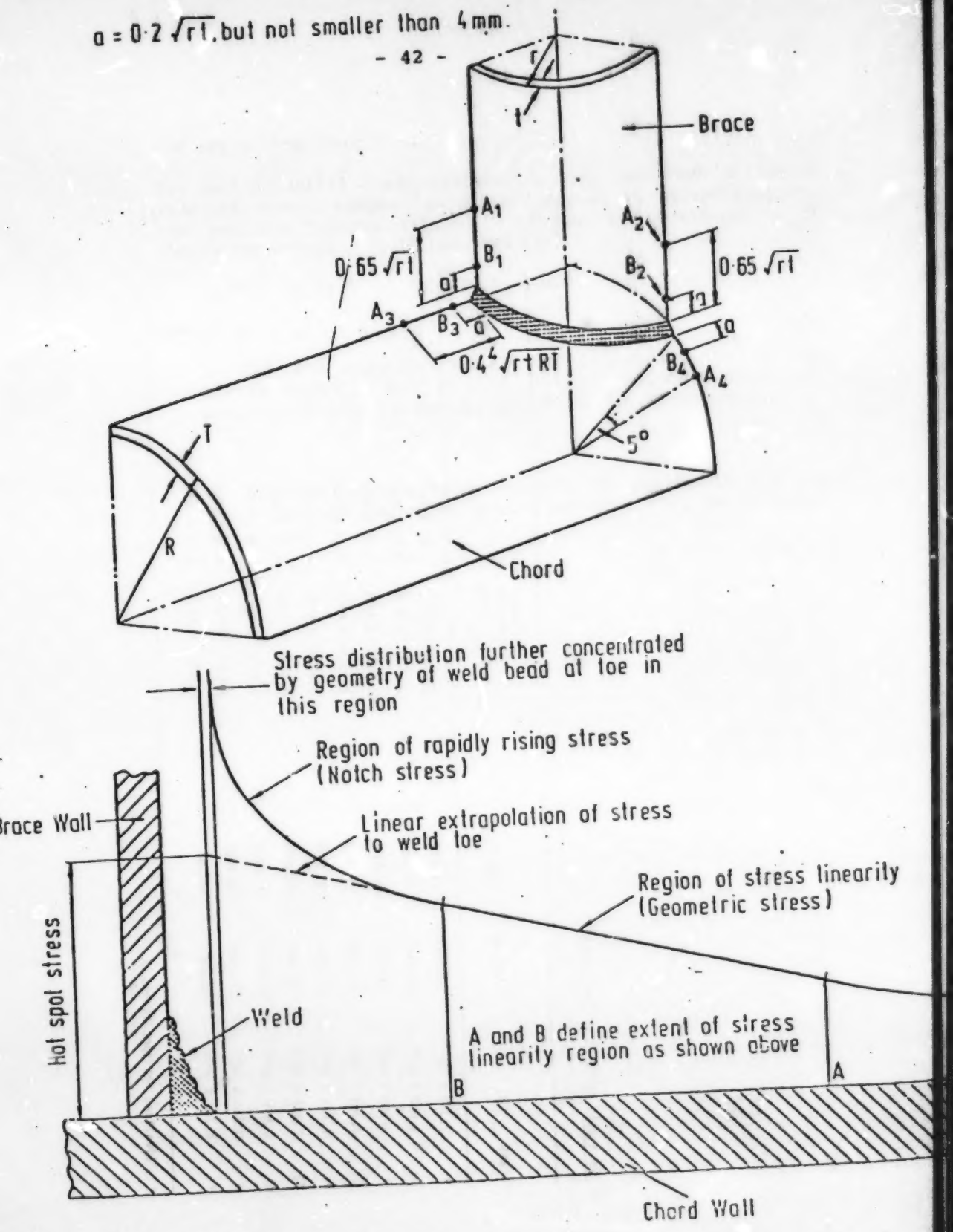


Fig. B. 1. Definition of idealized hot spot stress.

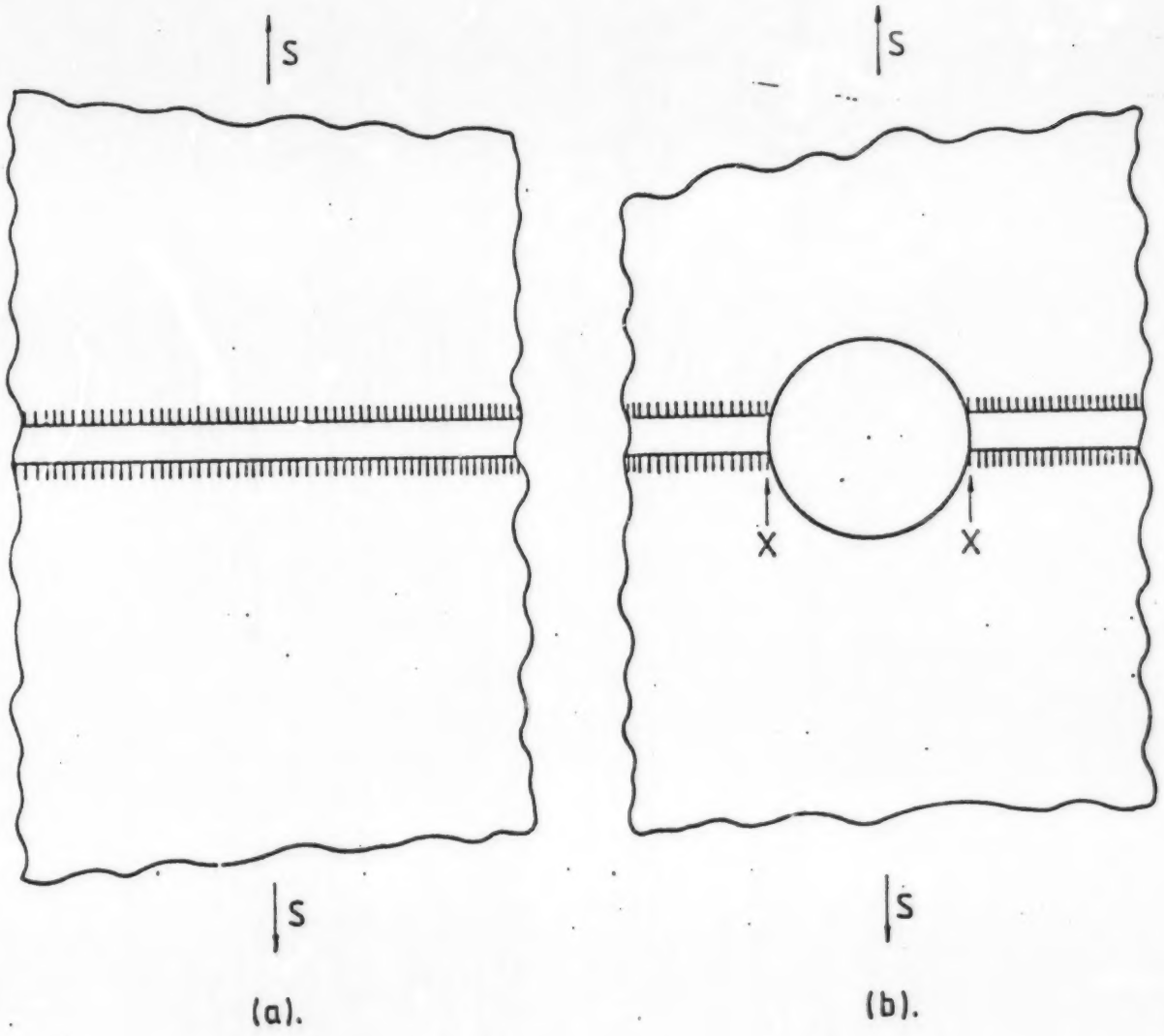
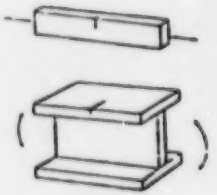


Fig. B.2. Explanation of nominal stress when weld is situated in region of stress concentration resulting from structure's gross shape.

Figure B.3 Classification of Joints

JOINT CLASSIFICATIONS

Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
TYPE 1 MATERIAL FREE FROM WELDING			
Notes on potential modes of failure In plain steel, fatigue cracks initiate at the surface, usually either at surface irregularities or at corners of the cross-section. In welded construction, fatigue failure will rarely occur in a region of plain material since the fatigue strength of the welded joints will usually be much lower. In steel with rivet or bolt holes or other stress concentrations arising from the shape of the member, failure will usually initiate at the stress concentration.			
1.1 Plain steel			
(a) With all surfaces machined and polished and of uniform or uniformly varying cross-section.	A		
(b) In the as-rolled condition, or with cleaned surfaces but with no flame-out edges or re-entrant corners.	B	Beware of using Class B for a member which may acquire stress concentrations during its life, e.g. as a result of rust pitting. In such an event Class C would be more appropriate.	
(c) As (b) but with any flame-cut edges subsequently ground or machined to remove all visible sign of the drag lines.	B	Any re-entrant corners in flame-out edges should have a radius greater than the plate thickness.	
(d) As (b) but with the edges machine flame-cut by a controlled procedure to ensure that the cut surface is free from cracks.	C	Note, however, that the presence of a re-entrant corner implies the existence of a stress concentration so that the design stress should be taken as the net stress multiplied by the relevant stress concentration factor.	

Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes

TYPE 2 CONTINUOUS WELDS ESSENTIALLY PARALLEL TO THE DIRECTION OF APPLIED STRESS

Notes on potential modes of failure

With the excess weld metal dressed flush, fatigue cracks would be expected to initiate at weld defect locations. In the as-welded condition, cracks might initiate at stop-start positions or, if these are not present, at weld surface ripples.

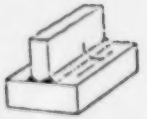
General comments

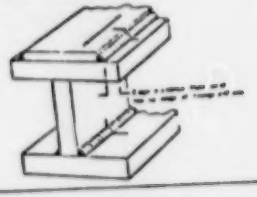
1. Backing strips

If backing strips are used in making these joints: (a) they must be continuous, and (b) if they are attached by welding those welds must also comply with the relevant Class requirements (note particularly that tack welds, unless subsequently ground out or covered by a continuous weld, would reduce the joint Class F, see type joint 6.5).

2. Edge distance

An edge distance criterion exists to limit the possibility of local stress concentrations occurring at unwelded edges as a result of example of undercut, weld spatter, or accidental overweave in manual fillet welding (see also notes on joint Type 4). Although an edge distance can be specified only for the 'width' direction of an element, it is equally important to ensure that no accidental undercutting occurs on the unwelded corners of, for example, cover plates or box girder flanges. If it does occur it should subsequently be ground smooth.

2.1	Full or partial penetration butt welds, or fillet welds Parent or weld metal in members without attachments, built up of plates or sections, and joined by continuous welds.			
(a)	Full penetration butt welds with the weld overfill dressed flush with the surface and finish-machined in the direction of stress, and with the weld proved free from significant defects by nondestructive examination.	B	The significance of defects should be determined with the aid of specialist advice and/or by the use of fracture mechanics analysis. The NDT technique must be selected with a view to ensuring the detection of such significant defects.	

Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
(b) Butt or filett welds with the welds made by an automatic submerged or open arc process and with no stop-start positions within the length.	C	If an accidental stop-start occurs in a region where Class C is required remedial action should be taken so that the finished weld has a simila- surface and root profile to that intended.	
(c) As (b) but with the weld containing stop-start positions within the length.	D	For situation at the ends of flange cover plates see joint type 6.4.	

TYPE 3 TRANSVERSE BUTT WELDS (i.e. essentially perpendicular to the direction of applied stress)

Notes on potential modes of failure

With the weld ends machined flush with the plate edges, fatigue cracks in the as-welded condition normally initiate at the weld toe, so that the fatigue strength depends largely upon the shape of the weld overfill. If this is dressed flush the stress concentration caused by it is removed and failure is then associated with weld defects. In welds made on a permanent backing strip, fatigue cracks initiate at the weld metal/strip junction, and in partial penetration welds (which should not be used under fatigue conditions), at the weld root.

Unless made on a permanent backing (Type 3.3) welds made entirely from one side are not classified for fatigue purposes, since adequate control cannot be exercised over the profile of the root bead which is where fatigue cracks would be likely to initiate.

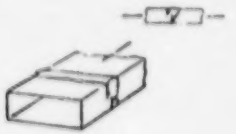
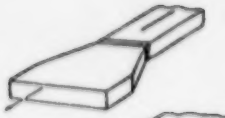
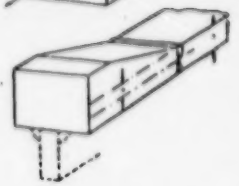
Design stresses

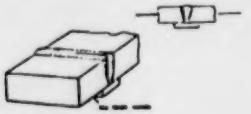
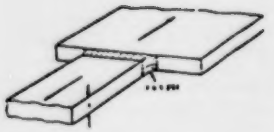
In the design of butt welds of Types 3.1 or 3.2 which are not aligned the stresses must include the effect of any eccentricity. An approximate method of allowing for eccentricity in the thickness direction is to multiply the nominal stress by $(1 + 3\,e/t)$, where

e is the distance between centres of thickness of the two abutting members; if one of the members is tapered, the centre of the untapered thickness must be used, and

t is the thickness of the thinner member.

With connections which are supported laterally, e.g. flanges of a beam which are supported by the web, eccentricity may be neglected.

Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
3.1 Parent metal adjacent to, or weld metal in, full penetration butt joints welded from both sides between plates of equal width and thickness or where differences in width and thickness are machined to a smooth transition not steeper than 1 in 4.		Note that this includes butt welds which do not completely traverse the member, such as circular welds used for inserting infilling plates into temporary holes.	
(a) With the weld overfill dressed flush with the surface and with the weld proved free from significant defects by non-destructive examination.	C	The significance of defects should be determined with the aid of specialist advice and/or by the use of fracture mechanics analysis. The NDT technique must be selected with a view to ensuring the detection of such significant defects.	
(b) With the welds made in the shop, either manually or by an automatic process other than submerged-arc, provided all runs are made in the flat position.	D	In general welds made positionally, or on site, or by the submerged-arc process tend to have a poor reinforcement shape, from the point of view of fatigue strength. Hence such welds are downgraded from D to E.	
(c) Welds made other than in (a) or (b)	E	In both (b) and (c) the corners of the cross-section of the stressed element at the weld toes should be dressed to a smooth profile. Note that step changes in thickness are, in general, not permitted under fatigue conditions, but that where the thickness of the thicker member is not greater than 1.15 x the thickness of the thinner member, the change can be accomodated in the weld profile without any machining. Step changes in width lead to large	

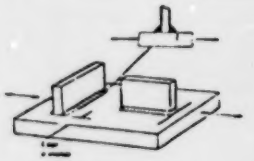
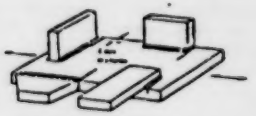
Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
		reductions in strength (see joint Type 3.3).	
3.2 Parent metal adjacent to, or weld metal in, full penetration butt welded joints made on a permanent backing strip between plates of equal width and thickness or with differences in width and thickness machined to a smooth transition not steeper than 1 in 4.	F	Note that if the backing strip is fillet welded or tack welded to the member the joint could be reduced to Class G (joint type 4.2).	
3.3 Parent metal adjacent to, or weld metal in, full penetration butt welded joints made from both sides between plates of unequal width, with the weld ends ground to a radious not less than 1.25 times the thickness t.	F2	Step changes in width can often be avoided by the use of shaped transition plates, arranged so as to enable butt welds to be made between plates of equal width. Note that for this detail the stress concentration has been taken into account in the joint classification.	

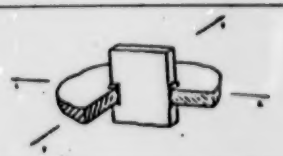
TYPE 4 WELDED ATTACHMENTS ON THE SURFACE OR EDGE OF A STRESSED MEMBER

Notes on potential modes of failure

When the weld is parallel to the direction of the applied stress fatigue cracks normally initiate at the weld ends, but when it is transverse to the direction of stressing they usually initiate at the weld toe; for attachments involving a single, as opposed to a double, weld cracks may also initiate at the weld root. The cracks then propagate into the stressed member. When the welds are on or adjacent to the edge of the stressed member the stress concentration is increased and the fatigue strength is reduced; this is the reason for specifying an 'edge distance' in some of these joints (see also note on edge distance in joint type 2).

4.1 Parent metal (of the stressed member) adjacent to toes or ends of bevel-butt or fillet welded attachments, regardless of the orientation of the weld to the direction of applied stress, and whether		Butt welded joints should be made with an additional reinforcing fillet so as to provide a similar toe profile to that which would exist in a fillet welded joint.	

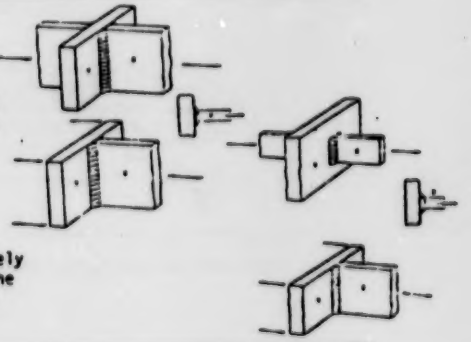
Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
or not the welds are continuous round the attachment.			
(a) With attachment length (parallel to the the direction of the applied stress) ≤ 150mm and with edge distance ≥ 10mm.	F	The decrease in fatigue strength with increasing attachment length is because more load is transferred into the longer gusset giving an increase in stress concentration.	
(b) With attachment length (parallel to the the direction of the applied stress) > 150mm and with edge distance ≥ 10mm.	F2		
4.2 Parent metal (of the stressed member) at the toes or the ends of butt or fillet welded attachments on or within 10mm of the edges or corners of a stressed member, and regardless of the shape of the attachment.	G	Note that the classification applies to all sizes of attachment. It would therefore include, for example, the junction of two flanges at right angles. In such situations a low fatigue classification can often be avoided by the use of a transition plate (see also joint type 2.3).	
4.3 Parent metal (of the stressed member) at the toe of a butt weld connecting the stressed member to another member slotted through it.		Note that this classification does not apply to fillet welded joints (see joint type 5.1b). However, it does apply to loading in either direction (L or T in the sketch).	
(a) With the length of the slotted-through member, parallel to the direction of the applied stress, ≤ 150mm and with edge distance ≥ 10mm.	F		

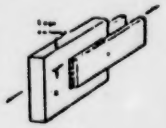
Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
(b) With the length of the slotted-through member, parallel to the direction of the applied stress, > 150mm and with edge distance ⩾ 10mm.	F2		
(c) With edge distance < 10mm.	G		

TYPE 5 LOAD-CARRYING FILLET AND T BUTT WELDS

Note on potential modes of failure

Failure in cruciform or T joints with full penetration welds will normally initiate at the weld toe, but in joints made with load-carrying fillet or partial penetration butt welds cracking may initiate either at the weld toe and propagate into the plate or at the weld root and propagate through the weld. In welds parallel to the direction of the applied stress, however, weld failure is uncommon; cracks normally initiate at the weld end and propagate into the plate perpendicular to the direction of applied stress. The stress concentration is increased, and the fatigue strength is therefore reduced, if the weld end is located on or adjacent to the edge of a stressed member rather than on its surface.

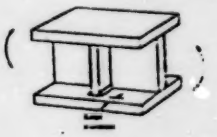
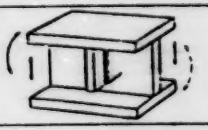
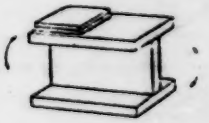
5.1 Joint description Parent metal adjacent to cruciform joints or T joints (member marked X in sketches).		Member Y can be regarded as one with a non-load-carrying weld (see joint type 4.1) Note that in this instance the edge distance limitation applies.	
(a) Joint made with full penetration welds and with any undercutting at the corners of the member dressed out by local grinding.	F		
(b) Joint made with partial penetration or fillet welds with any undercutting at the corners of the member dressed out by local grinding.	F2	In this type of joint failure is likely to occur in the weld throat unless the weld is made sufficiently large (see joint type 5.4).	

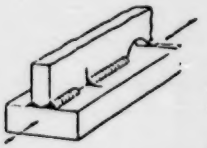
Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
5.2 Parent metal adjacent to the toe of load-carrying fillet welds which are essentially transverse to the direction of applied stress (member X in sketch).		The relevant stress in member X should be calculated on the assumption that its effective width is the same as the width of member Y.	
(a) Edge distance ≥ 10mm.	F2	These classifications also apply to joints with longitudinal welds only.	
(b) Edge distance < 10mm.	G		
5.3 Parent metal at the ends of load-carrying fillet welds which are essentially parallel to the direction of applied stress, with the weld end on plate edge (member Y in sketch).	G		
5.4 Weld metal in load-carrying joints made with fillet or partial penetration welds, with the welds either transverse or parallel to the direction of applied stress (based on nominal shear stress on the minimum weld throat area).	W	This includes joints in which a pulsating load may be carried in bearing, such as the connection of bearing stiffeners to flanges. In such examples the weld should be designed on the assumption that none of the load is carried in bearing.	

TYPE 6 DETAILS IN WELDED GIRDERS

Notes on potential modes of failure

Fatigue cracks generally initiate at weld toes and are especially associated with local stress concentrations at weld ends, short lengths of return welds, and changes of direction. Concentrations are enhanced when these features occur at or near an edge of a part (see notes on joint type 4).

Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
General comment			
Most of the joints in this section are also shown, in a more general form, in joint type 4; they are included here for convenience as being the joints which occur most frequently in welded girders.			
6.1 Parent metal at the toe of a weld connecting a stiffener, diaphragm, etc. to a girder flange.		Edge distance refers to distance from a free, i.e. unwelded, edge. In this example, therefore, it is not relevant	
(a) Edge distance ≥ 10mm (see joint type 4.2).	F	as far as the (welded) edge of the web plate is concerned. For reason for	
(b) Edge distance < 10mm.	G	edge distance see note on joint type 2.	
6.2 Parent metal at the end of a weld connecting a stiffener, diaphragm, etc. to a girder web in a region of combined bending and shear.	E	This classification includes all attachments to girder webs.	
6.3 Parent metal adjacent to welded shear connectors			
(a) Edge distance > 10mm	F		
(b) Edge distance < 10mm (see type 4.2)	G		
6.4 Parent metal at the end of a partial length welded cover plate, regardless of whether the plate has square or tapered ends and whether or not there are welds across the ends.	G	This Class includes cover plates which are wider than the flange. However, such a detail is not recommended because it will almost inevitably result in undercutting of the flange edge where the transverse weld crosses it, as well as involving a longitudinal weld terminating on the flange edge and causing a high stress concentration.	

Type number, description and notes on mode of failure	Class	Explanatory Comments	Examples, including failure modes
6.5 Parent metal adjacent to the ends of discontinuous welds, e.g. intermittent web/flange welds, tack welds unless subsequently buried in continuous runs.	E	This also includes tack welds which are not subsequently buried in a continuous weld. This may be particularly relevant in tack welded backing strips.	
Ditto, adjacent to cope holes.	F	Note that the existence of the cope hole is allowed for in the joint classification; it should not be regarded as an additional stress concentration.	

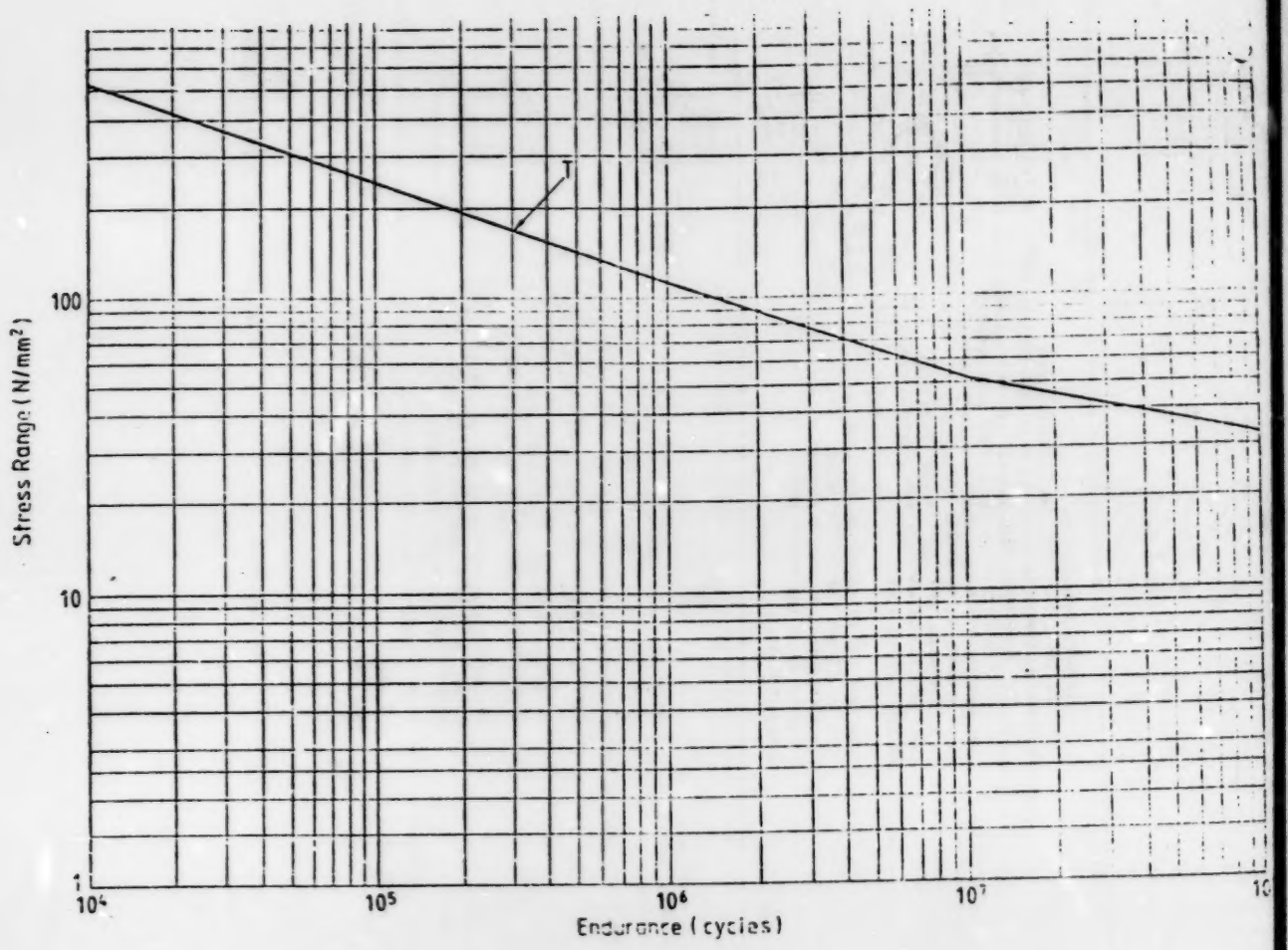


Fig. B.4. S - N Curves for model joints.

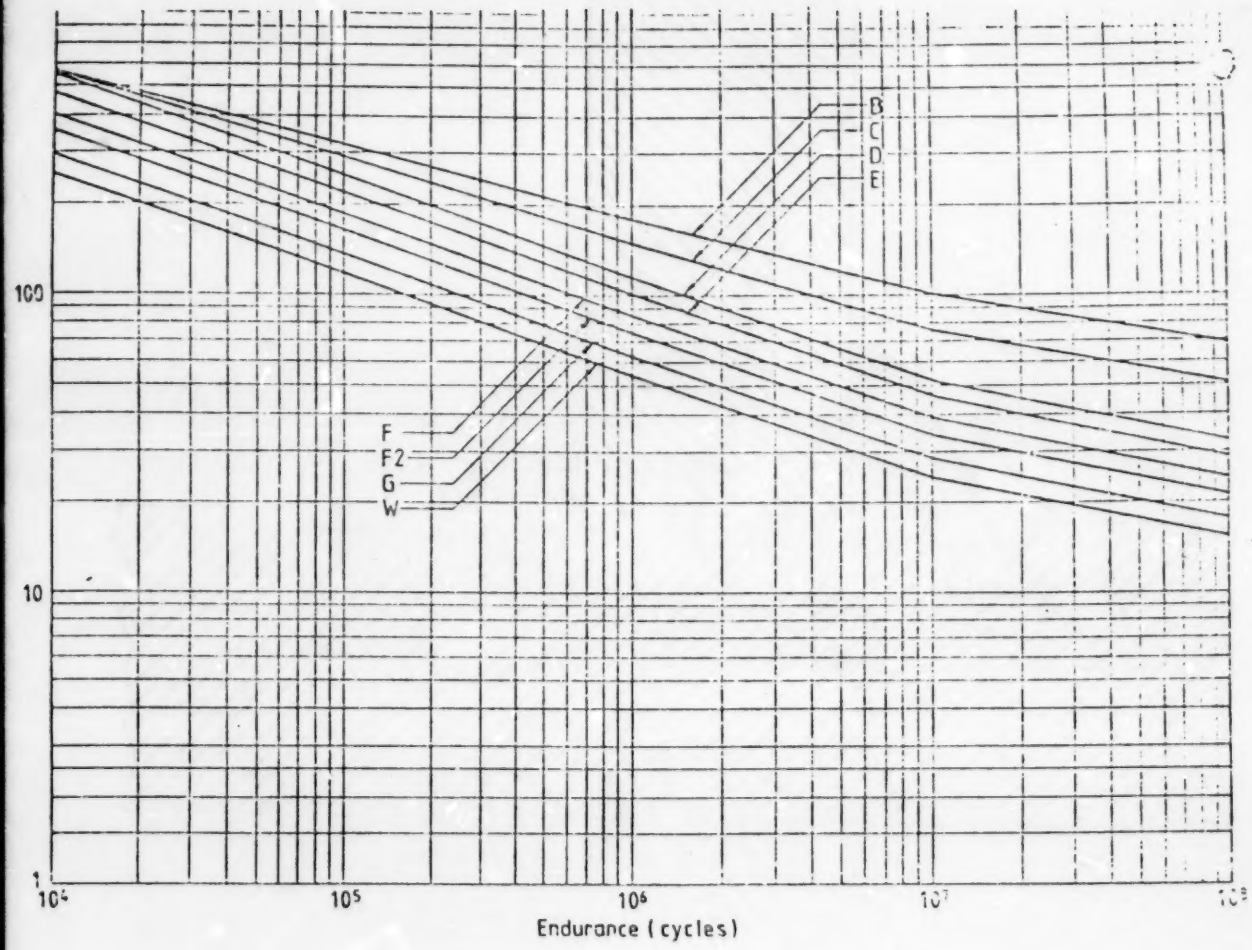


Fig. B.5. S-N Curves for non-nodel joints.

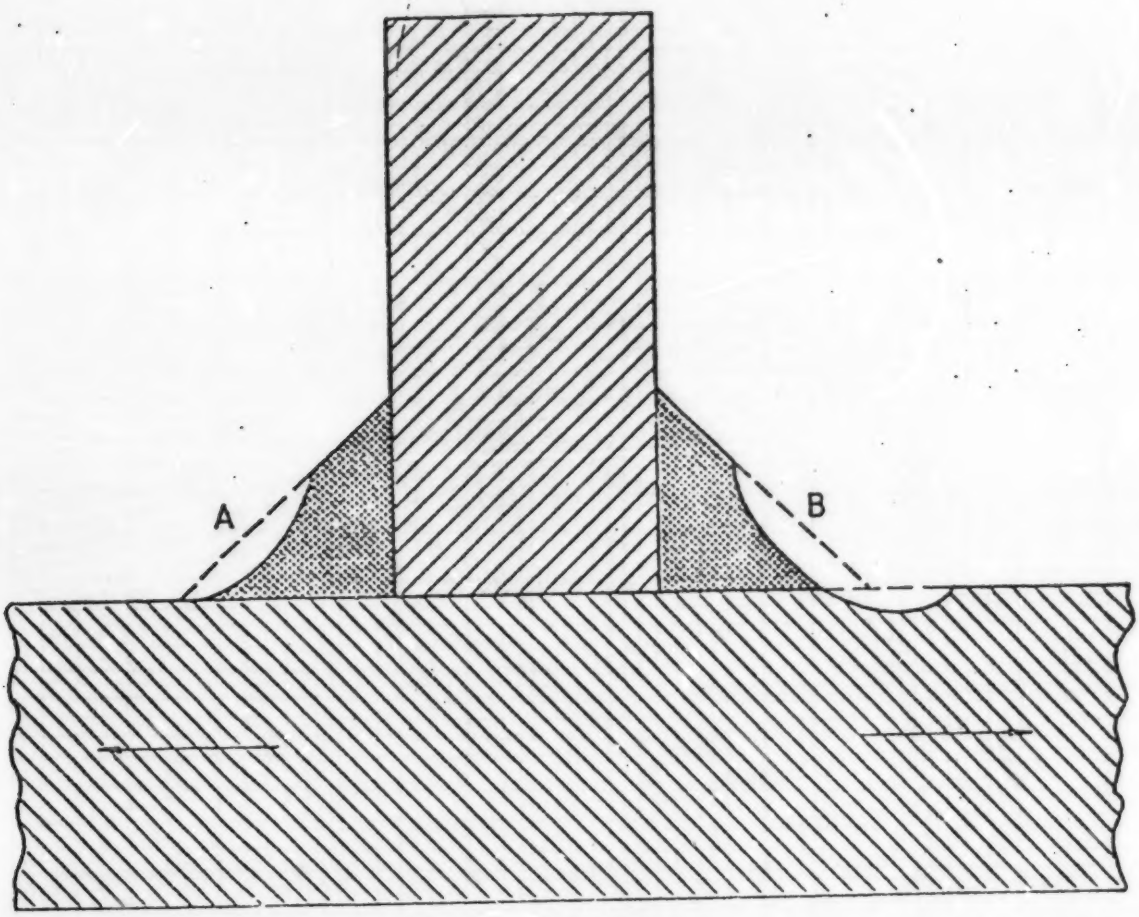


Fig. B.6. Grinding a weld toe tangentially to the plate surface, as at A, will produce little improvement in strength. Grinding must extend below the plate surface, as at B, in order to remove toe defects.

APPENDIX C: GROUTED PILE-SLEEVE CONNECTIONS

APPENDIX C: GROUTED PILE-SLEEVE CONNECTIONS

C.1 STRENGTH OF A GROUTED CONNECTION

(a) The specified bond strength of a grouted connection with or without mechanical shear connectors, satisfying the requirements of C.2 is given by:

$$f_{buc} = K.C_L\ (9C_s + 1100\ h/s)\ (f_{cu})^{1/2} \quad (1)$$

where

f_{buc} is the specified bond strength (in N/mm^2)

f_{cu} is the specified grout compressive strength (in N/mm^2) (see C.4)

K is the stiffness factor defined below

C_L is the coefficient for grouted length to pile diameter ratio

C_s is the surface condition factor

h is the minimum shear connector outstand (in mm)

s is the nominal shear connector spacing (in mm)

(b) The stiffness factor K is defined by:

$$K = 1/m\ (D/t)_g^{-1} + [(D/t)_p + (D/t)_s]^{-1} \quad (2)$$

where m is the modular ratio of steel to grout

D is the outside diameter

t is the wall thickness

suffixes g, p and s refer to grout, pile and sleeve respectively.

(c) In the absence of other data the modular ratio m may conservatively be taken as 18 for long term (i.e. 28 days or more).

(d) The available data on the parameter C_L are limited. In the absence of data relating to the specific tubular and

shear connector geometry, the following values of C_L should be assumed:

L/D_p	C_L
2	1.0
4	0.9
8	0.8
>12	0.7

where L is the nominal grouted connection length

Intermediate values for $L/D_p < 12$ should be calculated by linear interpolation.

(e) The surface condition factor C_s should be taken according to the following:

i) if shear connectors are present and satisfy the requirement $h/s > 0.005$, the C_s may be taken as 1.0.

ii) for plain pipe connections and for connections with shear connectors but with $h/s < 0.005$, then in the absence of test data C_s should be taken as 0.6.

The values in (i) and (ii) above refer to shotblasted or lightly rusted surface conditions. Other conditions (e.g. painted surfaces) should receive speci_l consideration.

C.2 RANGE OF APPLICATION

The relationship given in C.1 may only be applied to connections which satisfy the following geometrical ratio limits.

Sleeve geometry	$50 < (D/t)_s < 140$
Pile geometry	$24 < (D/t)_p < 40$

Grout annulus geometry	$10 < (D/t)_g < 45$
Grouted connection length to pile diameter ratio	$L/D_p > 2$
Shear connector height ratio	$0^* < h/D_p < 0.006$
Shear connector spacing ratio	$0^* < D_p/s < 8$
Shear connector ratio	$0^* < h/s < 0.04$
Shear connector shape factor	$1.5 < w/h < 3$

where w is the nominal width of the shear connector including welds.

* refers to plain pipe connections

The notation is illustrated in Figure C.1.

The above limits are imposed because of the range of geometries for which test data are available. The restriction on shear connector shape factor (width/outstand) is consistent with connectors formed either by welded square bar or approximately hemispherical weld beads and generally covers current practice. These limitations should not be taken to exclude the use of connections outside the stated ranges providing that it is demonstrated that the strength of such connections can be reliably estimated. Connections with pile and sleeve D/t ratios below the lower limits (i.e. more radially stiff) can be conservatively designed by assuming the limiting D/t ratios in calculations. Alternatively test data may be used as the basis for design. However, care must be taken in using the results of limited test programmes to provide a specified bond strength equivalent to that given in C.1. Statistical analysis of test data requires the imposition of substantial reduction factors in the calculation of specified bond strength values from a small amount of data.

C.3 SAFETY FACTORS

The following factors should be applied to the specified bond strength given by C.1.

(a) Grout displacing water

Condition	Safety factor
Extreme	4.5
Normal/Operating	6.0

(b) Grout displacing drilling mud or other similar material

Condition	Safety factor
Extreme	6.0
Normal/Operating	8.0

C.4 ASSESSMENT OF COMPLIANCE OF GROUT STRENGTH

C.4.1 GROUT SPECIMENS

The formula (1) for bond strength given in C.1 is based on the specified grout compressive strength as determined by tests on 75mm cubes at the design age (usually 28 days). Other cube sizes and cylinders may be used to determine and check grout strength providing a conversion factor is applied as determined by appropriate tests. It should be noted that for any given mix the conversion factor will depend on the age of the grout.

Testing of grout specimens to determine grout compressive strength should be in accordance with reference 1.

C.4.2 MIX SPECIFICATION

Evidence that the proposed mix will meet the specified strength at a given age should be obtained from previous production data or trial mixes. The mean strength calculated from these data should exceed the specified specified grout compressive strength by either

(i) $1.64\ \sigma\ (0.86 + 2/n)$ for $10 < n < 100$

or (ii) $1.64\ \sigma$ for $n > 100$

where σ is the standard deviation calculated from n results

n is the number of test results not less than 10.

Previous production data for use in the above criteria should be test results from separate grout batches selected over an immediately prior period not exceeding two years, using the materials and plant which are proposed for the work.

Where laboratory trial mixes are used to provide data at least three separate batches of grout should be prepared and at least six cylinders should be made from each batch for each age at which compliance is to be determined.

C.4.3 OFFSHORE QUALITY CONTROL

During the grouting of each connection samples of grout should be taken from randomly selected batches. The rate of sampling should take account of the nature of the work.

At least four samples each of 3 specimens (cubes/cylinders) should be taken for each connection. One specimen from each sample should be tested to assess compliance (usually at 28 days). The remaining specimens may be tested at earlier ages to indicate the grout quality.

The specimens should be subjected, until test, to a curing regime representative of the curing conditions of the grouted connection, i.e. underwater and at the appropriate seawater temperature.

Strength compliance should be assumed if no test result in each set of 4 is below the specified grout compressive strength. In the event of non-compliance, the action taken should have due regard for the kind of degree of non-compliance and the implication on safety.

C.5 OFFSHORE PRACTICES

For the design relationships given in C.1 to be applicable the grout should be injected at the lowest point of the annulus so that any fluid present is completely displaced. Reliable means of ensuring complete filling of the annulus with sound grout should be incorporated, such as provision for samples of material at the top of the annulus to be returned to the surfaces by piping or by divers, or the provision of proven remote monitoring devices at the top of the annulus.

C.6 REFERENCE

1. BS 1881

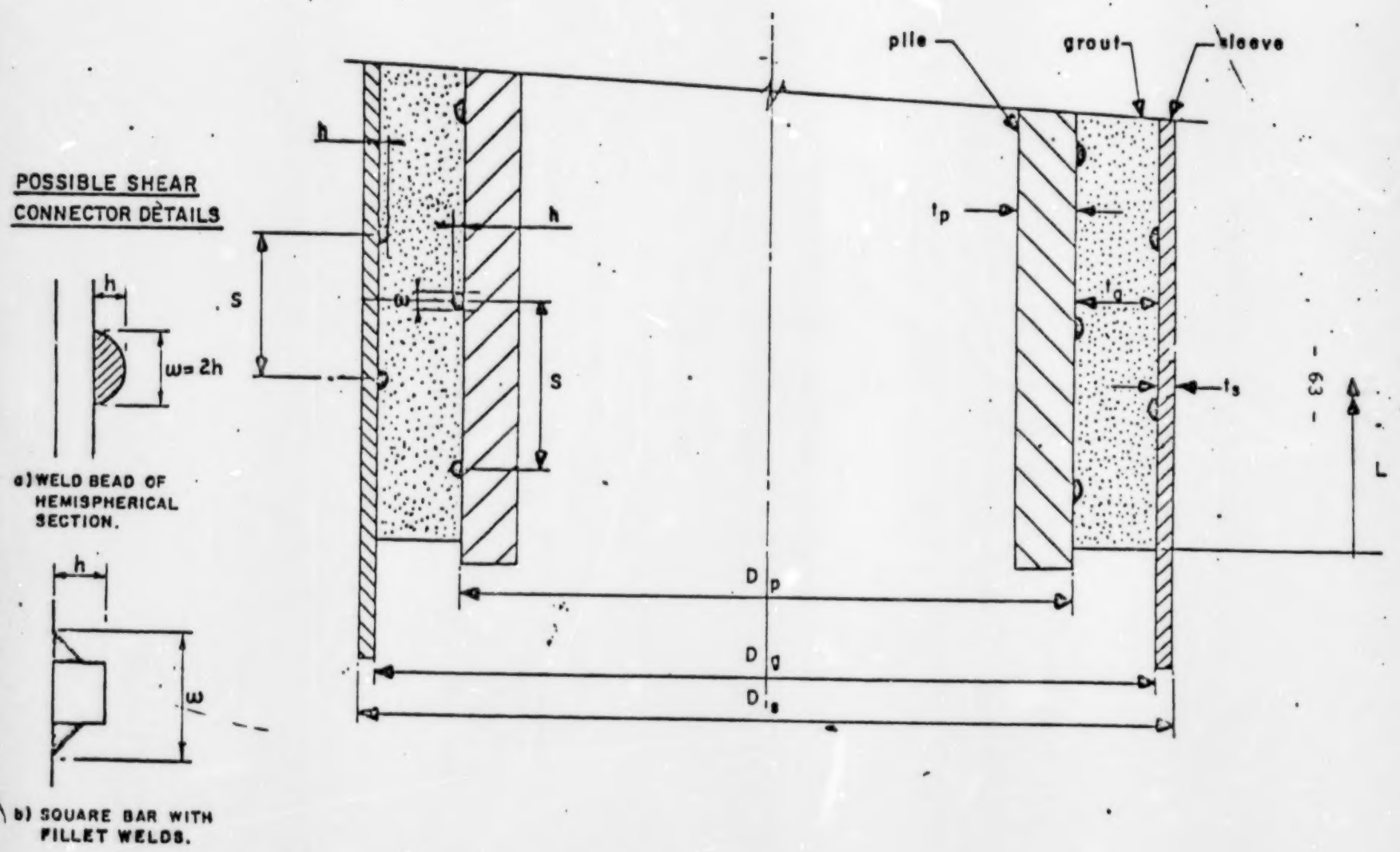


Fig. C.1. Grouted pile/sleeve connection.

APPENDIX D: NOISE AND VIBRATION LIMITS

APPENDIX D: NOISE AND VIBRATION LIMITS

D.1 NOISE

D.1.1 SPECIFIC WORK AREA NOISE LIMITS

The recommended noise limits for particular work areas such as control rooms etc. are given in Table D.1.

TABLE D.1 Recommended noise limits for specific work areas of offshore installations

Specific work areas of offshore installations	noise limit, dBA
Workshops	70
General stores	70
Kitchens	60
Control rooms	55
Offices	55
Laboratories	55
Radio/Communication rooms	45

D.1.2 SLEEPING/RECREATION AREA NOISE LIMITS

The recommended noise limits for areas of living accommodation on offshore installations are given in Table D.2. These limits should be regarded as maximum acceptable noise levels for these areas. Where lower levels can be relatively easily obtained (eg. by ventilation silencing) then it is desirable that these noise control measures should be implemented.

TABLE D.2 Recommended noise limits for sleeping/recreation areas of living accommodation on offshore installations

Sleeping/Recreation areas of living accommodations	Noise limit, dBA
Washing Facilities	60
Changing rooms	60
Toilets	60
Dining rooms	55
Recreation rooms	50
Theatre/Meeting rooms	45
Television rooms	45
Sleeping areas	45
Medical rooms	45
Quiet rooms	45

Note: Noise levels in corridors should not generally be more than 5 dBA greater than the noise limits in adjoining rooms, with a maximum level of 60 dBA in any corridor.

D.1.3 RELAXATION OF NOISE LIMITS

D.1.3.1 General Work Area

All reasonably practicable means should be taken to comply with the specified noise levels. If the limit cannot be achieved in certain areas then these should be treated as 'restricted' and appropriate warning notices posted. Personnel entering these areas should generally be obliged to wear suitable ear protectors unless their daily unprotected noise exposure can be shown to be within the requirements of ACGIH standards.

Examples of restricted areas could be as follows:

(a) Normal operational conditions: within noise control enclosures large enough to admit service personnel.

(b) Intermittent operational conditions: on the helideck when the helicopter is present.

(c) Design emergency conditions: near emergency safety relief valves.

D.1.3.2 All Other Areas

The noise limits specified for these areas should normally apply at all times although in certain circumstances this may not be practical, eg. during helicopter movements.

D.1.4 NOISE RATING NUMBERS

The octave band values corresponding to the noise rating curves are given in Table D.3.

TABLE D.3 Noise rating numbers and corresponding octave band sound pressure levels approximating to dBA noise criteria

Noise Criteria, dBA	Design Noise rating number	Octave band centre frequency, Hz 63	125	250	500	1K	2K	4K	8K
90	NR 85	103	96	91	88	85	83	81	80
88	NR 83	101	94	89	86	83	81	79	78
70	NR 65	87	79	72	68	65	63	61	59
60	NR 55	79	70	63	58	55	52	50	49
55	NR 50	75	56	59	54	50	47	45	44
50	NR 45	71	61	54	49	45	42	40	38
45	NR 40	67	57	49	44	40	37	35	33

Note: Where alternative dBA noise criteria are specified the appropriate NR values should be selected.

D.2 VIBRATION

D.2.1 STANDARDS AND DEFINITIONS

D.2.1.1 Terminology

Interpretation of the terminology used in these recommendations should be in accordance with International Standard ISO 2041, Vibration and Shock Vocabulary.

D.2.1.2 Relevant Surfaces For Application of Criteria

The recommended vibration limits should be met on surfaces designated as normal access for standing and sitting.

D.2.2 VIBRATION LIMITS FOR HUMAN EXPOSURE

D.2.2.1 Exposure Time

The vibration limits recommended for general work areas are based on a 12-hour working day. These limits should be taken as the design values for offshore installations. In certain circumstances some relation of the general work area vibration limits may be considered acceptable.

D.2.2.2 Frequency Range

The specified criteria cover the frequency range 1-80Hz and are not intended to be extrapolated beyond these limits.

D.2.2.3 Direction of Vibration

Criteria are specified for both vertical and horizontal linear motion and apply separately to the resolved components of the vibration in these directions. Direction, a_z, should be taken to correspond with the foot to head axis of the human body (reference 1).

D.2.2.4 Non-sinusoidal, Random or Broad Band Vibration

Such vibration should be analyzed using narrow band filters with a bandwidth not exceeding one-third octave (reference 2). The limits are then appropriate to the centre frequency of that band.

In cases where the crest factor (ratio of peak to r.m.s.) is greater than 3, the recommended limits should be regarded as tentative.

In certain circumstances vibration components at more than one frequency, occurring simultaneously, may interact, and a weighted r.m.s. summation of the interaction components may be necessary (reference I).

D.2.2.5 Criteria

The recommended vibration limits are divided into five different categories detailed in Table D.4, ranging from recommended vibration limits in living accommodation (category V) to prohibited vibration levels (greater than category I). The vibration limits are shown in Figures D.1 and D.2.

TABLE D.4 Description of vibration limit categories for offshore installations. This table should be read in conjunction with Figures D.1 and D.2.

Category	Description
I	Restricted area (less than 4 minutes exposure) vibration limits. Short exposure to levels above these limits may create a health hazard and cause difficulty in walking. These high levels of vibration usually cause such alarm and discomfort that action is immediately and intuitively taken by persons subject to the vibration. Vibration levels above these limits should be treated as prohibited.
II	Just acceptable locally to equipment, although vibration limits for machinery may be more restrictive than these levels. Annoyance and discomfort may be experienced.
III	Recommended design vibration limits for all general work areas. Vibration levels are easily detectable but not unconformable.
IV	Recommended design vibration limits for office, control rooms and similar areas.
V	Recommended design vibration limits for sleeping, recreation and similar areas in living accommodation. These vibration levels are just detectable.

D.2.3 RELAXATION OF VIBRATION LIMITS

All reasonably practicable means should be taken in the design of offshore installations to achieve vibration levels in general work areas equal to or less than the limits given in category III. Higher levels than those given in category II may, however, be tolerated for shorter exposure periods than 12 hours without a serious hazard to health, although some fatigue and decreased working proficiency may occur. The maximum allowable exposure period for levels of vibration greater than given in category II may be calculated from Table D.5. Note that the existence of these higher levels is not recommended and every effort should be made to limit the levels to those given in category III. Where exposure occurs to varying levels of high vibration over the period of the working day, then an equivalent exposure ratio may be calculated in accordance with the method given in Reference 1 with reference to the maximum levels given in category I. If the equivalent exposure ratio exceeds unity then the exposure should be classified as prohibited.

No relaxation of vibration limits is recommended for categories IV and V.

TABLE D.5 Restricted exposure periods to high vibration levels.

a_t/a_1	Allowable exposure time, t
1.00	1-4 minutes
0.92	8 minutes
0.76	15 minutes
0.59	30 minutes
0.42	1 hour
0.29	2 hours
0.18	4 hours
0.11	8 hours
0.08	12 hours

Note: (i) : t is the allowable time for vibration level, a_t

(ii) : a is the restricted area vibration limit (category I) for 1-4 minutes' exposure.

D.3 REFERENCES

1. British Standard Institution. Guide to the evaluation of human explosure to whole body vibration. BSI DD 32.

2. British Standards Institution. Octave and one-third octave band-pass filters. BS 2475.

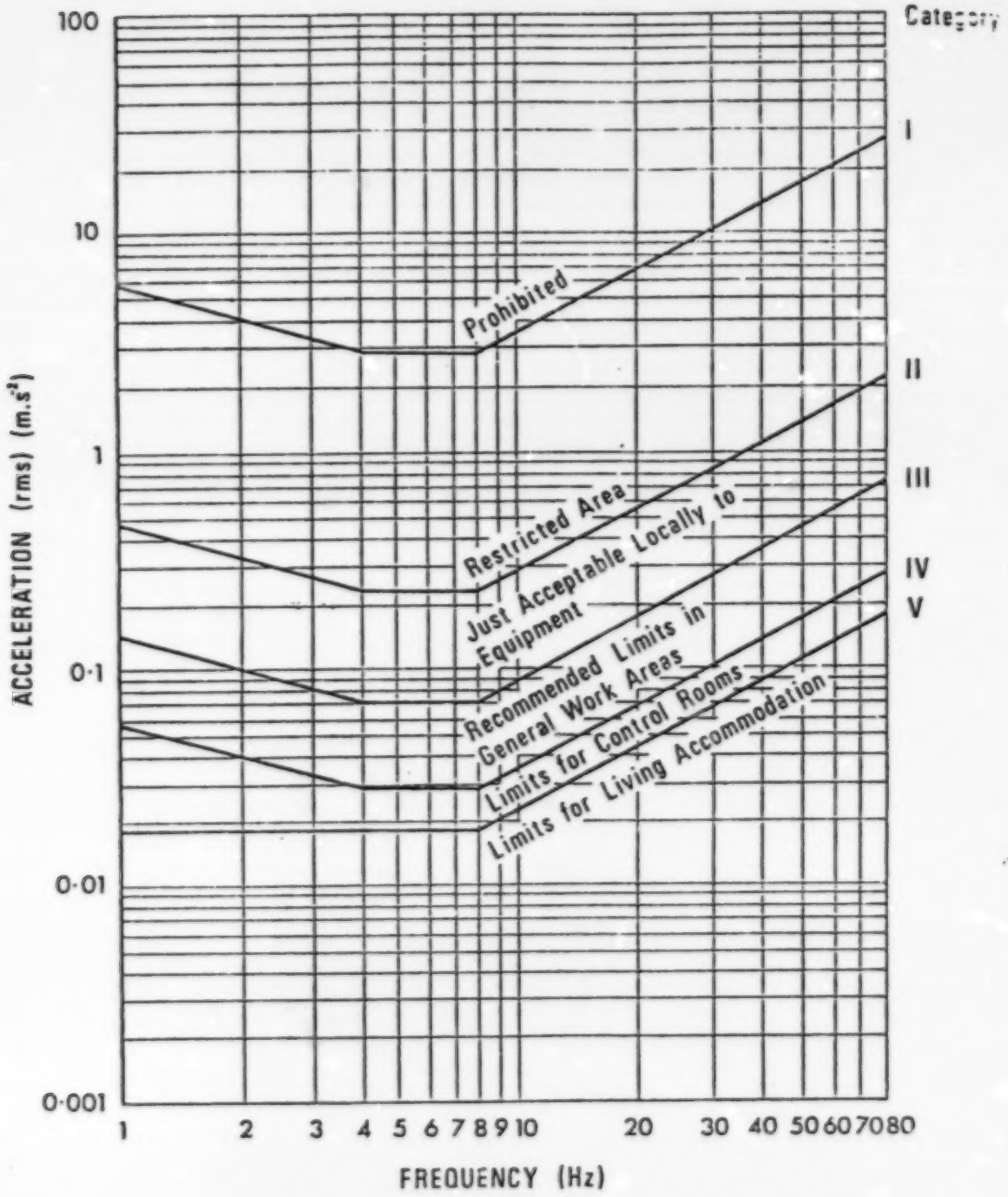


Fig. D.1. Vibration limits for offshore installations 'vertical' axis (A_z).

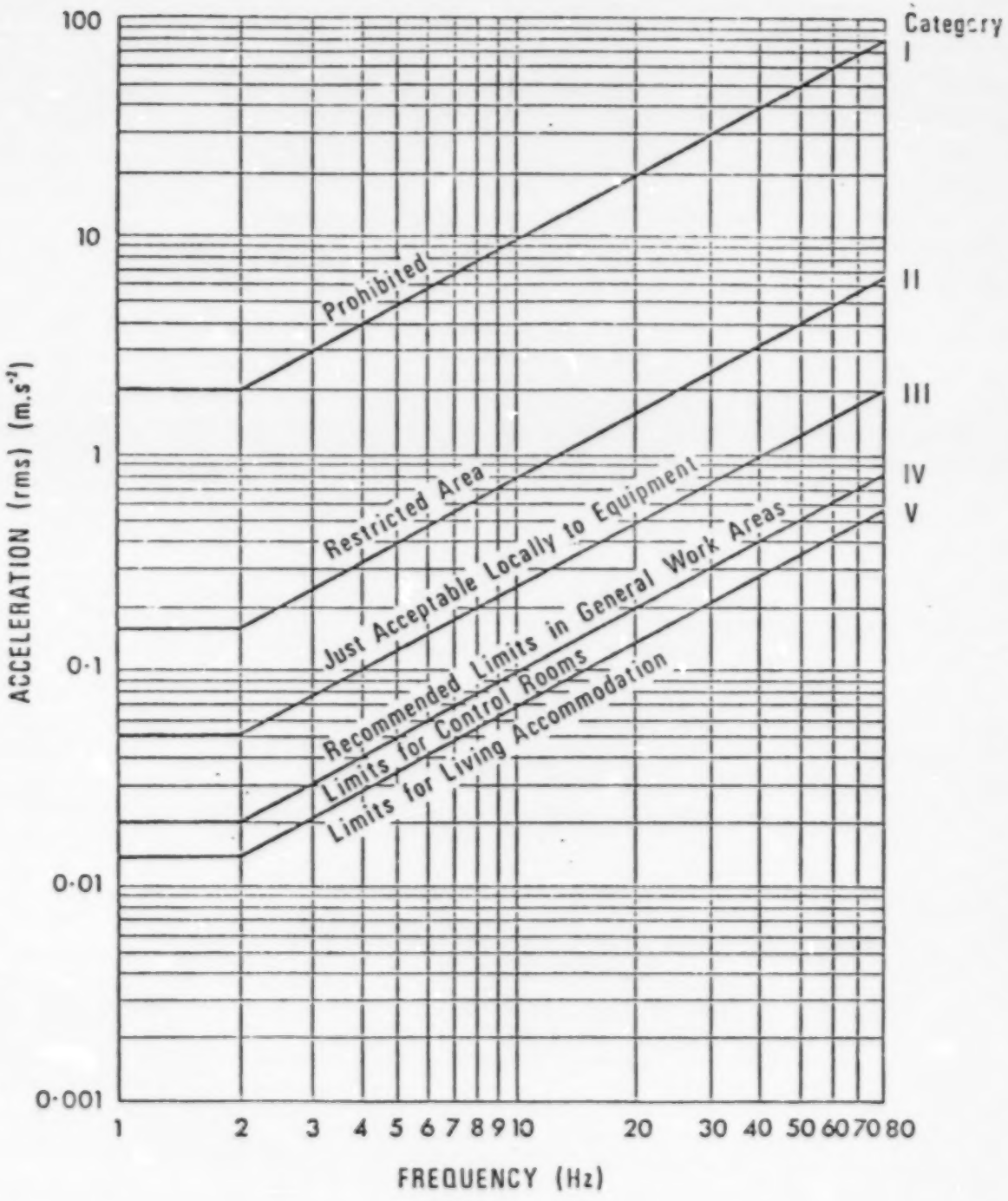


Fig. D.2. Vibration limits for offshore installations, 'horizontal' axes (A_x, A_y)

APPENDIX E: NOTCH TOUGHNESS

APPENDIX E: NOTCH TOUGHNESS

E.1 DESIGN VALUES

The minima values for notch toughness of steel are given in Table E.1 and Figure E.1. With heavy sections and high strengths it may be necessary to adopt more severe requirements than are indicated in the table.

(a) For fixed installations steels 3, 4, 7 and 8 (Table E.1) should be used for parts of the primary structure above 6 metres below low spring tide level which are subject to particularly arduous stress conditions, for example, nodes in tubular constructions, heavy sections, joints in main girders and other critical connections. Steels 1, 2, 5 and 6 (Table E.1) may be used for all other parts of the primary structure.

(b) For mobile installations, where appropriate to the nature of the structure, Table E.1 should be used as guidance on the choice of steels. Where the nature of the structure is such that shipbuilding practice is more appropriate, ship practice should be followed.

(c) The impact tests should be carried out on specimens taken from just below the surface of the plate (reference 1). In cases where a full evaluation of weld metal properties is required (eg. in thicker plates) it is recommended that specimens also be taken from mid-thickness. All specimens should be taken from welds laid down with a procedure and heat input corresponding to that used in practice and should be cut, transverse to the weld with the notch perpendicular to the plate surface.

(d) It is recommended that for steels requiring impact testing every plate should be tested.

(e) The energy requirements for Charpy V notch tests in accordance with Figure E.1 are given in Table E.1. The energy requirement for weld metal is 27J.

For higher strength materials, the energy requirement should be agreed with the certifying authority.

E.2 REFERENCES

1. British Standars Institution. Methods for notched bar tests. BS 131.

TABLE E.1 Minima values for notch toughness

Reference	Strength level Maximum value of minimum specified			Charpy V notch test requirements (before stress relieving)		
Steel	Yield stress N/mm^2	Tensile strength N/mm^2	Thickness mm	Energy joules	Temperature °C Not to be stress relieved	To be stress relieved
1 Mild steels for	280	430	<25	27	- 5	
2 structures (as rolled;	280	430	>25	27	-20	-10
3 controlled rolled;	280	430	<20	27	-20	
4 normalized)	280	430	>20	27	-40	-30
5 High tensile steels	355	500	<25	34	- 5	
6 for structures	355	500	>25	34	-20	-10
7 (controlled rolled;	355	500	<20	34	-20	
8 normalized; Q and T)	355	500	>20	34	-40	-30

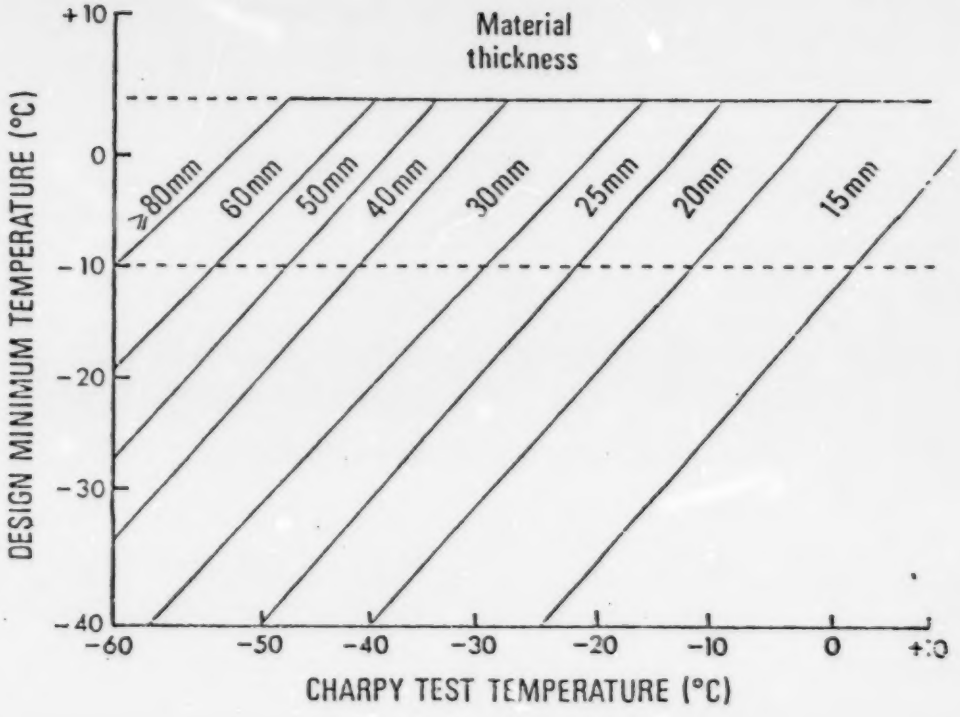


Fig. E.1. Minima values for notch toughness (as-welded condition)

APPENDIX F: HELICOPTER LANDING AREA

APPENDIX F: HELICOPTER LANDING AREA

F.1 HELICOPTER WEIGHTS AND DIMENSIONS

The weights, dimension and landing net sizes for different types of helicopters are given in Table F.1

Table F.1 Helicopter Weights and Dimensions

Helicopter	Overall Length "D" Meters	Maximum Weight kg.	Roter Height (approx.) Meters	Landing Net Size
Bolkow Bo 105	11.05	2300	3	small
Dauphin 2.SA 365	13.29	3400	3.5	-
Bell 222*	14.41	3446	3.25	-
Sikorsky S.76	16.00	4347	3	-
Bell 212	17.46	5080	4.5	medium
Puma SA 330	18.18	7400	4.25	medium
Sikorsky S.58T	20.06	5896	4.5	large
Wessex 60	20.08	6170	4.5	large
Sikorsky S.61N	22.20	9298	4.5	large
Sikorsky S.65	26.90	19051	5.25	-
Boeing 234LR	30.18	21315	2.5/5.68	-

(Chinook)* (Tandem Main Rotor)

F.2 SIZE AND OBSTACLE ENVIRONMENT

The Certificate of Fitness states for each helicopter landing area the maximum size 'D' for which that area is considered suitable and is qualified by the type of helicopter, single main rotor helicopters or tandem main rotor helicopters.

For any particular type of single main rotor helicopter the landing area should be sufficiently large to contain a circle of diameter 'D' equal to the overall length of that helicopter measured across the main and tail rotors in a fore-and-aft line. For any helicopter with tandem-main rotors a measurement across the main rotors in the fore-and-aft line is taken as 'D'. A rectangular landing area for such helicopters should be sufficiently large to provide a longitudinal axis of at least 0.9 'D' and an athwartships axis of at least 0.75 'D'. The criteria for the two types are shown in Figures F.1 and F.2.

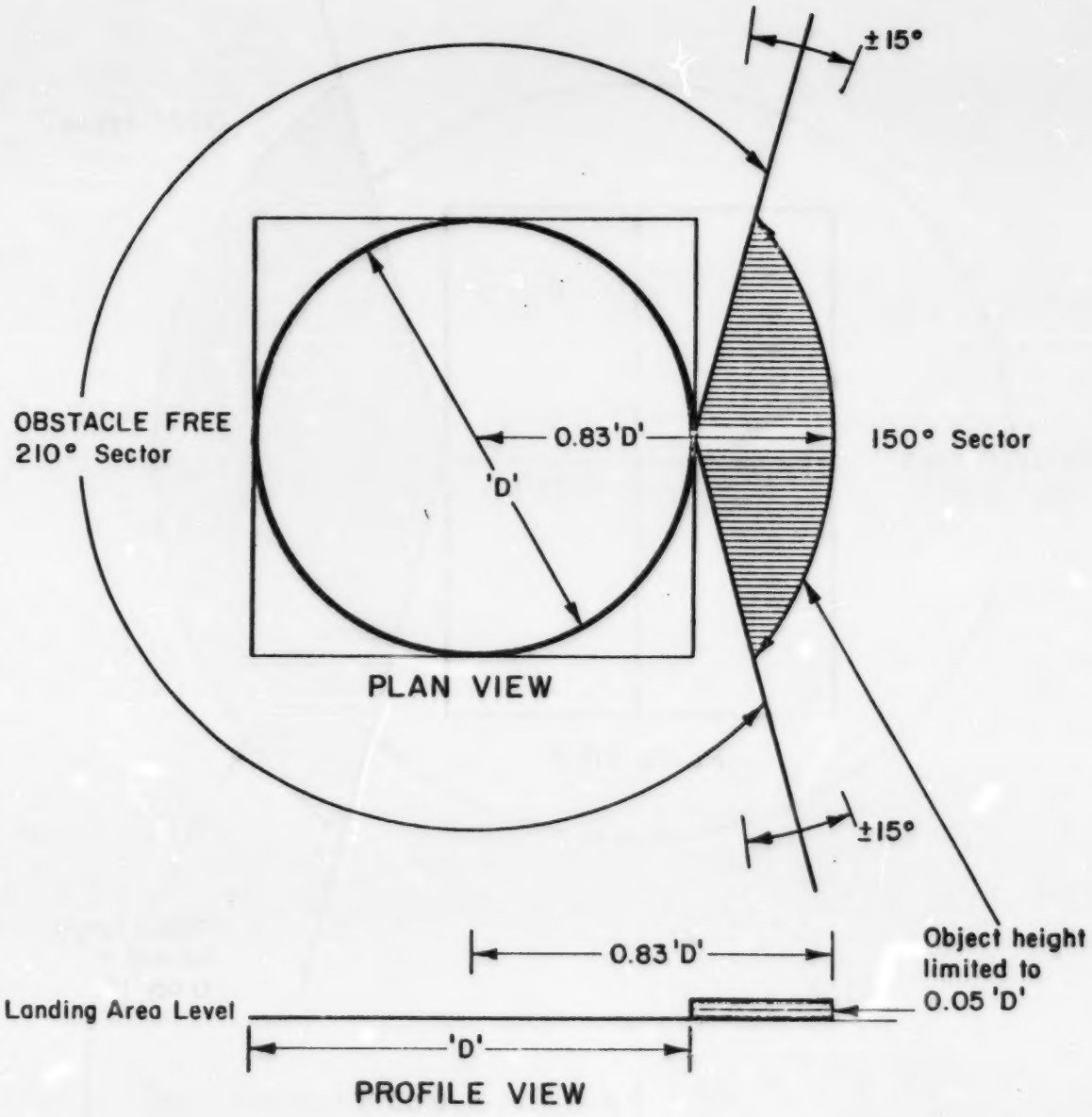


Fig. F.1. Obstacle free areas - single main rotor helicopters

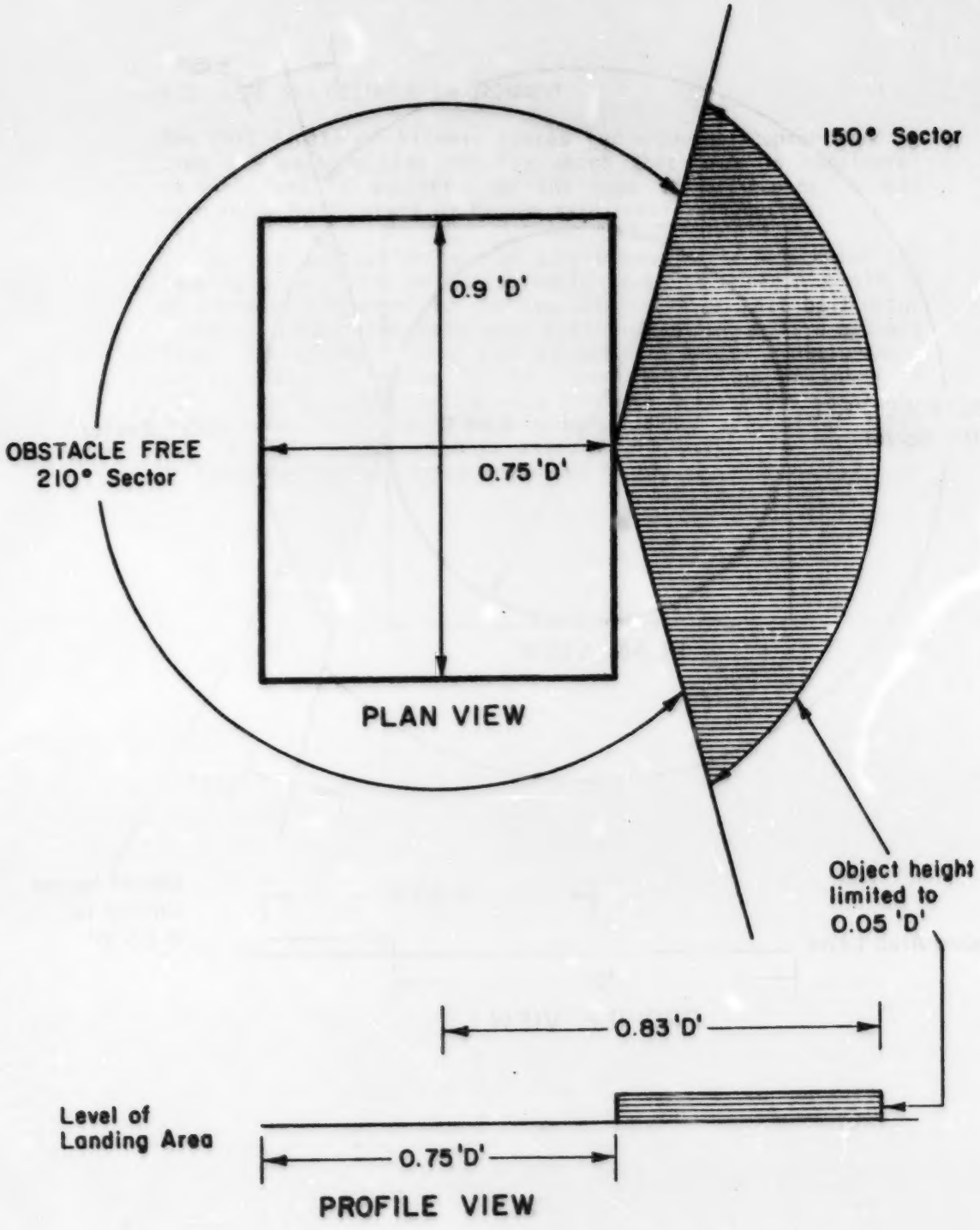


Fig. F.2. Obstacle free areas - tandem - main rotors helicopters

POOR

QUALITY

ORIGINAL